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16. Abstract

The Soils and Foundations Workshop manual is directed to bridge and foundation engineers, particularly those involved in design and construction aspects of a highway project. The manual is geared to the practicing engineer who routinely deals with soils and foundations problems but has little theoretical background in soil mechanics or foundation engineering. The manual content follows a project-oriented approach where foundation work is traced from preparation of the boring request through design computation of settlement, allowable footing pressure, etc., to the construction of approach embankments, pile driving operations, etc. Recommendations are presented on how to layout borings efficiently, how to minimize approach embankment settlement and eliminate the bump at the end of-the bridge, how to design the most cost-effective pile foundation, and how to transmit design information properly to construction through plans, specifications, or contact with the project engineer.

The objective of this workshop manual is to present a recommended method for safe, cost-effective design and construction of foundations. Coordination between engineers in all project phases is stressed. Readers are encouraged to develop an appreciation of foundation activities in all project phases which influence or are influenced by their work

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# PREFACE

The Soils and Foundations Workshop is designed for bridge and foundation engineers involved in the preliminary layout, design, or construction aspects of a highway project. This manual is intended to serve both as the workbook for the course and later as a reference notebook on foundations. The material contained in this book is geared to the practicing engineer in the foundation field who routinely deals with soil and foundation problems but has little theoretical background in soil mechanics or foundation engineering.

The manual follows a project oriented approach whereby the soils input to a fictitious bridge project is traced from conception to completion in a serialized illustrative workshop design problem.

The concepts presented in each chapter are concise and specifically directed at a particular operation in the foundation design process. Basic examples are included in several sections for hands-on knowledge. Continuity between chapters is achieved by sequencing the information in the normal progression of a foundation design study. In each phase of the fictitious project the soil concepts are developed into specific foundation designs or recommendations for that segment of the workshop design problem.

## ACKNOWLEDGEMENTS

Permission by the FHWA to adapt the July 1993 Soils and Foundations Workshop Manual prepared by Richard Cheney and Ronald Chassie (FHWA) is gratefully acknowledged. Provision by the FHWA of the electronic files and hard copy of exhibits is also acknowledged. Furthermore the efforts of Jerry DiMaggio, Andy Muńoz, Ivan Marreo, Chris Dumas, and Peter Osborn of the FHWA are acknowledged for their thorough technical reviews and recommendations.

The author would also like to acknowledge the assistance of Jeremy Hung, Roger Gobin, and a number of professionals from Parsons Brinckerhoff Quade and Douglas, Inc., in the preparation of this revision of the manual. Many thanks to Theresa Kong for overall compiling, Roger Gobin for overseeing the preparation and production, and Theresa Kong, Moises Young, and Sandor Aguilar for graphic preparation and word processing.

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## SOILS AND FOUNDATIONS WORKSHOP

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## Symbols

Chapter 2

| | |
|---|---|
| ASTM | American Society for Testing and Materials |
| AASHTO | American Association of State Highway and Transportation Officials |
| ASCE | American Society of Civil Engineers |
| RQD | Rock Quality Designation |
| SPT | Standard Penetration Test |
| N | SPT blows per foot |
| $C_u$ | Undrained shear strength |
| $\phi$ | Angle of friction |

Chapter 4

| | |
|---|---|
| W | Moisture content |
| $\gamma$ | Unit weight |
| n | Porosity |
| e | Void ratio |
| $G_S$ | Specific gravity |
| $\mu$ | Pore water pressure |
| $P_T$ | Total overburden pressure |
| $P_o$ | Effective overburden pressure |
| $\gamma_b$ | Effective unit weight |
| $\gamma_w$ | Unit weight of water |
| $\gamma_T$ | Total unit weight |
| LL | Liquid Limit |
| PL | Plastic Limit |
| PI | Plastic Index |
| SL | Shrinkage Limit |
| SI | Shrinking Index |
| Ac | Activity |
| LI | Liquidity Index |
| $C_c$ | Compression Index |
| $P_c$ | Preconsolidation Pressure |
| $C_r$ | Recompression Index |
| $e_o$ | Initial Void Ratio |
| $C_v$ | Coefficient of Consolidation |
| $C_\alpha$ | Coefficient of Secondary Compression |
| S | Shear Strength |
| C | Cohesion |
| N | Normal Force |
| $\phi$ | Angle of internal friction (total) |
| U | Unconfined Compression |
| UU | Unconsolidated Undrained Triaxial |
| CU | Consolidated Undrained Triaxial |
| CD | Consolidated Drained Triaxial |
| DS | Direct Shear |
| | |
| $D_{10}$ | Gradation characteristics effective diameter (10% by weight of sample is finer than this |

| | |
|---|---|
| | diameter) |
| $D_{30}$, $D_{60}$, $D_{85}$ | Percent grain size by weight of sample finer than 30, 60, 85% |
| $C_F$ | Coefficient of uniformity |
| $C_z$ | Gradation characteristics, Coefficient of curvature |
| $a_v$ | Consolidation characteristics: coefficient of compressibility |
| $m_v$ | Coefficient of volume compressibility |
| $C_s$ | Swelling Index |
| $\phi'$ | Angle of internal friction (effective) |
| c' | Cohesion intercept (effective) |
| $q_u$ | Unconfined compressive strength |
| $S_t$ | Sensitivity |
| $E_s$ | Modulus of elasticity |
| $\gamma_{max}$ | Maximum dry unit weight |
| OMC | Optimum moisture content |
| $D_d$ | Relative density |
| CBR | California bearing ratio |

Chapter 5

| | |
|---|---|
| C | Cohesion |
| $\gamma_{Fill}$ | Fill soil unit weight |
| $H_{Fill}$ | Fill height |
| S | Total shear strength |
| $\sigma$ | The total normal stress against the failure surface slice base due to the weight of soil and water above the failure surface |
| $\mu$ | Water pressure on the slice base |
| $\phi$ | Angle of internal friction |
| F.S. | Factor of Safety |
| $L_w$ | Level arm distance to the center of rotation |
| $L_s$ | Radius of circle |
| R | Moment arm |
| N | Effective normal force against the slice base (force between granular soil grains) |
| $W_t$ | Total slice weight |
| $\alpha$ | Angle between vertical and line drawn from circle center to midpoint of slice base |
| l | Arc length of slice base |
| $P_a$ | Active Force (Driving) |
| $P_p$ | Passive Force (Resisting) |
| $\gamma$ | Soil unit weight |
| H | Height of soil layer in active wedge |
| $K_a$ | Active earth pressure coefficient for level ground surface |
| $K_p$ | Passive earth pressure coefficient for level ground surface |
| $\alpha_w$ | Slope of water table from horizontal |
| UU | Unconsolidated Undrained Triaxial |
| CU | Consolidated Undrained Triaxial |
| CD | Consolidated Drained Triaxial |

Chapter 6

| | |
|---|---|
| b | Horizontal distance from embankment centerline to midpoint of slope |
| $G_s$ | Specific gravity |
| h | Embankment height |
| $\gamma_f$ | Unit weight of fill |
| $\Delta H$ | Settlement |
| H | Thickness of soil layer considered |
| C′ | Bearing capacity index |
| $P_o$ | Existing effective overburden pressure |
| $\Delta P$ | Distributed embankment pressure |
| $P_F$ | Final pressure felt by foundation subsoil |
| N | Standard Penetration Test value |
| N’ | Corrected SPT N value |
| $P_c$ | Preconsolidation pressure |
| $e_o$ | Initial void ratio at $P_o$ |
| $C_c$ | Compression indice |
| $C_r$ | Recompression indice |
| $\Delta H_{sec}$ | Secondary settlement |
| $C_\alpha$ | Coefficient of secondary consolidation (determined from lab consolidation test) |
| $t_{sec}$ | Time over which secondary settlement is being estimated |
| $t_p$ | Time for primary consolidation |
| T | Theoretical time factor |
| $H_v$ | Maximum length of vertical drainage path |
| $C_v$ | Coefficient of consolidation |
| K | Pressure Coefficient |
| $U_c$ | Percent consolidation combined radial and vertical |
| $U_R$ | Percent consolidation radial |
| $U_v$ | Percent consolidation vertical |
| $(_b$ | Bouyant unit weight |
| TR | Radial time factor |
| $d_C$ | Effective sand drain diameter |
| $d_N$ | Sand drain diameter |
| S | Center to center spacing of sand drains |
| $t_{90}$ | Time for 90%of primary consolidation |

Chapter 7

| | |
|---|---|
| N | SPT value |
| $q_{ult}$ | Ultimate capacity |
| $\gamma$ | Unit Weight |
| D | Footing embedment |
| $B_w$ | Footing width |
| $N_c$, $N_q$, $N_\gamma$ | Bearing capacity factor |
| X | Depth below footing |
| $d_o$ | Depth of influence of footing |
| f | Lateral extent of influence of footing |
| N’ | Corrected SPT value |
| $\phi$ | Angle of internal friction |
| C | Cohesion strength |
| $q_{all}$ | Allowable bearing capacity |

| | |
|---|---|
| $T$ | Theoretical time factor |
| $H_v$ | Maximum length of vertical drainage path |
| $t_{90}$ | Time for 90% of primary consolidation |
| $L$ | Footing length |
| $P$, $V$ | Applied footing pressure |
| $\Delta H$ | Settlement |
| $H$ | Thickness of soil layer considered |
| $C'$ | Bearing capacity index |
| $P_o$ | Existing effective overburden pressure |
| $\Delta P$ | Distributed footing pressure |
| $P_F$ | $P_o + \Delta P$ |
| $P_c$ | Preconsolidation pressure |
| $e_o$ | Initial void ratio |
| $C_c$ | Compression indices |
| $C_r$ | Recompression indices |

Chapter 8

| | |
|---|---|
| $\gamma$ | Soil unit weight |
| $Q_{drive}$ | Driving resistance |
| $Q_{ult}$ | Ultimate bearing capacity |
| $Q_s$ | Total skin resistance |
| $Q_p$ | Total point resistance |
| $D$ | Pile length below ground |
| $C_d$ | Pile perimeter |
| $N'$ | SPT value corrected for overburden pressure |
| $K_\delta$ | Dimensionless factor relating normal stress and effective overburden pressure |
| $P_d$ | Effective overburden pressure at the center of depth increment d |
| $\omega$ | Angle of pile taper measured from the vertical |
| $\delta$ | Friction angle on the surface of sliding |
| $d$ | Depth increment below ground surface |
| $C_F$ | Correction factor for $K_\delta$ when $\delta \neq \phi$ (soil friction angle) |
| $Q_p$ | Total end bearing |
| $A_p$ | Pile end area |
| $\alpha$ | Dimensionless factor dependent on depth-width relationship |
| $N'_q$ | Bearing capacity factor |
| $C_a$ | Pile adhesion |
| $C_u$ | Undrained shear strength |
| $P_{ult}$ | Group capacity |
| $n$ | Number of group piles |
| $E$ | Group efficiency |
| $C_h$ | Horizontal coefficient of consolidation. |
| $T$ | Time factor |
| $H_v$ | Maximum vertical drainage path in the clay layer(s) below the pile tips |
| $C_V$ | Coefficient of consolidation. |
| $Q_T$ | Total axial capacity of the drill shaft |
| $Q_B$ | Drill shaft base capacity |
| $Q_S$ | Drill shaft side capacity |
| $q_s$ | Capacity of pile segment (skin friction) |
| $\phi$ | Angle of internal friction |

| | |
|---|---|
| V | Volume per foot for pile segment |
| UU | Unconsolidated Undrained Triaxial |

Chapter 9

| | |
|---|---|
| W | Weight of the ram |
| H | Distance of ram fall |
| R | Total soil resistance (driving capacity) against the pile |
| S | Pile penetration (set) per blow |
| P | Safe pile load in kips |
| k | Constant which varies from 0.1 to 1 based on hammer type |
| E | Manufacturer rated energy (foot-pounds) at the stroke observed in the field |
| Log (10N) | Logarithm to the base 10 of the quantity 10 multiplied by N, the number of hammer blows per inch at final penetration (blows per inch) |
| Fy | Yield strength of steel |
| F′c | 28 day concrete cylinder strength |
| F′a | Allowable compressive stress of timber including allowance for treatment effects |
| X | Estimated pile footage |

**CONVERSION FACTORS**

| Approximate Conversions to SI Units | | | Approximate Conversions from SI Units | | |
|---|---|---|---|---|---|
| When you know | Multiply by | To find | When you know | Multiply by | To find |
| (a) Length | | | | | |
| inch | 25.4 | millimeter | millimeter | 0.039 | inch |
| foot | 0.305 | meter | meter | 3.28 | foot |
| yard | 0.914 | meter | meter | 1.09 | yard |
| mile | 1.61 | kilometer | kilometer | 0.621 | mile |
| (b) Area | | | | | |
| square inches | 645.2 | square millimeters | square millimeters | 0.0016 | square inches |
| square feet | 0.093 | square meters | square meters | 10.764 | square feet |
| acres | 0.405 | hectares | hectares | 2.47 | acres |
| square miles | 2.59 | square kilometers | square kilometers | 0.386 | square miles |
| (c) Volume | | | | | |
| fluid ounces | 29.57 | milliliters | milliliters | 0.034 | fluid ounces |
| gallons | 3.785 | liters | liters | 0.264 | gallons |
| cubic feet | 0.028 | cubic meters | cubic meters | 35.32 | cubic feet |
| cubic yards | 0.765 | cubic meters | cubic meters | 1.308 | cubic yards |
| (d) Mass | | | | | |
| ounces | 28.35 | grams | grams | 0.035 | ounces |
| pounds | 0.454 | kilograms | kilograms | 2.205 | pounds |
| short tons (2000 lb) | 0.907 | megagrams (tonne) | megagrams (tonne) | 1.102 | short tons (2000 lb) |
| (e) Force | | | | | |
| pound | 4.448 | Newton | Newton | 0.2248 | pound |
| (f) Pressure, Stress, Modulus of Elasticity | | | | | |
| pounds per square foot | 47.88 | Pascals | Pascals | 0.021 | pounds per square foot |
| pounds per square inch | 6.895 | kiloPascals | kiloPascals | 0.145 | pounds per square inch |
| (g) Density | | | | | |
| pounds per cubic foot | 16.019 | kilograms per cubic meter | kilograms per cubic meter | 0.0624 | pounds per cubic feet |
| (h) Temperature | | | | | |
| Fahrenheit temperature (°F) | 5/9(°F- 32) | Celsius temperature (°C) | Celsius temperature (°C) | 9/5(°C)+ 32 | Fahrenheit temperature (°F) |

Notes: 1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second(s), newton (N) and pascal (Pa=N/m$^2$).
2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

# CHAPTER 1.0
# INTRODUCTION

## 1.1 PURPOSE AND SCOPE

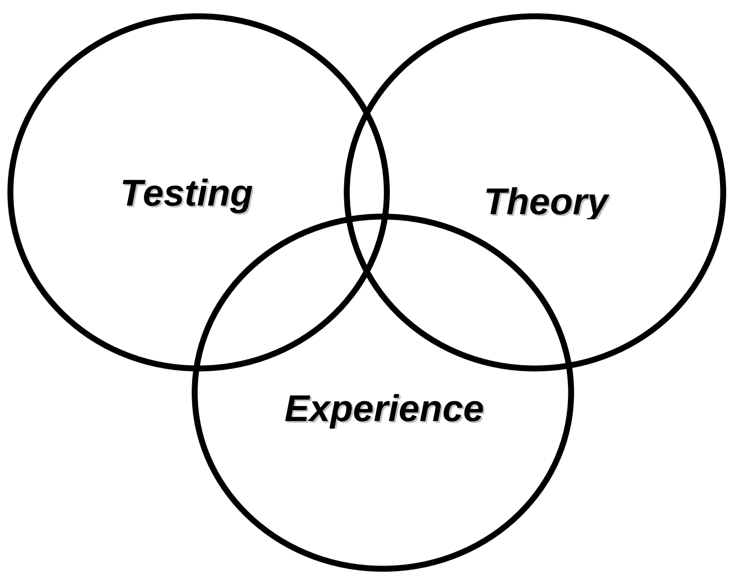
The Soils and Foundations Workshop is a 4-day training course sponsored by National Highway Institute to provide practical knowledge in geotechnical and foundation engineering for both generalists and those planning to take more advanced geotechnical courses in the future. The workshop is designed for bridge and foundation engineers involved in the design and construction aspects of a highway project.

This reference manual is the third edition of the Federal Highway Administration Soils and Foundations Workshop manual. The first edition was prepared in 1988 and a second edition with minor modifications came out in 1993. The manual is geared to the practicing engineer who routinely deals with soils and foundations problems but has little theoretical background in soil mechanics or foundation engineering. The overall goal of this manual is to present a recommended method for safe, cost-effective design and construction of foundations. Coordination between engineers in all project phases is stressed. The reader is encouraged to develop an appreciation of foundation activities in all project phases which influence or are influenced by his work

The manual follows a project oriented approach whereby the soils input to a fictitious bridge project is traced from conception to completion in a serialized illustrative workshop design problem.

## 1.2 SOILS AND FOUNDATIONS FOR HIGHWAY STRUCTURES

Man's earliest attempts at construction probably involved soil. As civilization developed through many centuries, man learned by trial and error about soil as a foundation material. Since World War I, much understanding of soil behavior has been achieved by applying the principles of physics, mechanics, hydraulics, strength of materials, and structural engineering. This approach to analyzing soils problems is called "soil mechanics." Because soil is a very complex medium, an entirely theoretical solution of most soil problems is not practical. The most practical solution to soil problems can be reached by a combination of the following sources of information.



1. Experience obtained by trial and error in the past; this developed into the empirical or "rule of thumb" procedures for today. The weakness of this approach is not recognizing differences in the engineering properties of soils. What works well at one location may not succeed with the same type of soil at another location.

2. Testing to obtain information on the properties of soils; generally obtained by field explorations and laboratory tests. Subsequent, theoretical analysis results will only be as good as the soils data used as input.

3. Theory based on scientific principles from various fields of engineering and science; used to explain or predict the behavior of soils under various conditions.

Analysis of soil is more complex than the analysis of other construction materials. Steel and concrete are relatively uniform solids which have predictable strength properties within the elastic range of loading. The strength may be "ordered" in the manufacture of steel and in the making of a concrete mix. This strength will be constant under all climatic conditions. Structures can then be built of these materials with confidence in their strength.

Soils deposits are composed of a mixture of three dissimilar materials; soil, water, and air. The soils' properties will be influenced by the action of each of these materials in the soil mass. Some of the factors influencing the strength of soil are:

1. Size, shape, and distribution of soil particles,

2. Degree of packing of soil particles,

3. Amount of water in soil, and

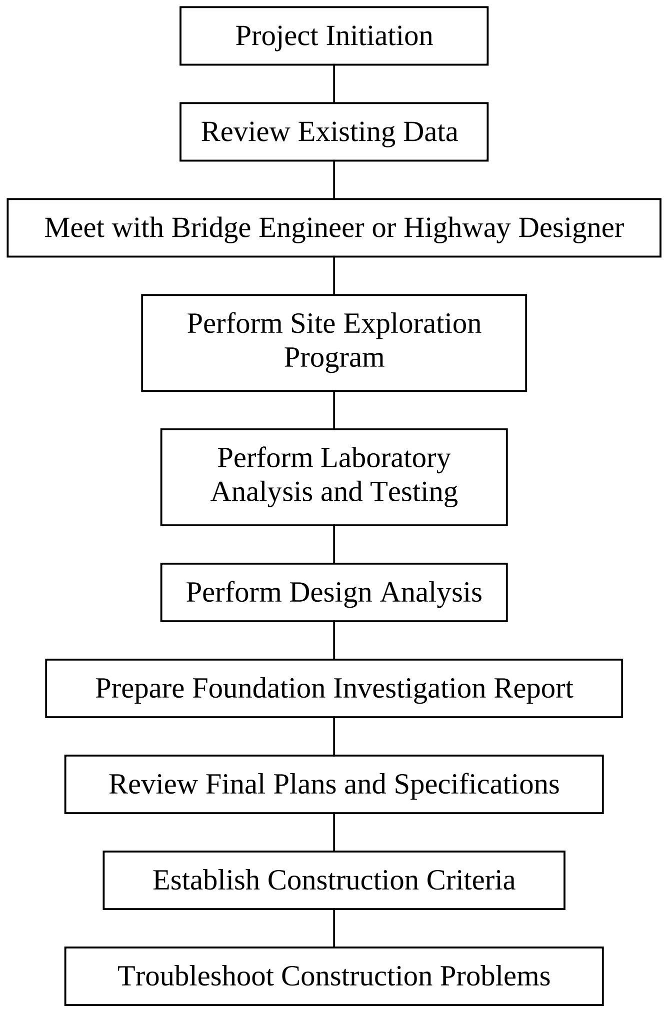
4. Climatic variations

Engineers should understand the fundamental properties of soils to use them as construction materials.

The success or failure of a foundation design is often decided in the early stages of a project. To assure success, the input of an experienced geotechnical engineer should begin at project inception and continue until completion of construction. The early interaction of the geotechnical engineer with other engineers will prevent establishment of a project alignment or grade which may require expensive foundation treatment later in design. It is imperative that good communication and interaction exist between the geotechnical engineer, structural engineer, and construction engineer, throughout the design and construction process to insure cost-effective design and to minimize design and construction problems. The importance of this communication and interaction will be stressed throughout this manual and cannot be overemphasized.

The following flow chart of geotechnical activities generally describes this involvement. A more specific listing of these activities for structure foundations is shown in Table 1, Geotechnical Involvement in Project Phases.

Flow Chart




## 1.3 ORGANIZATION OF MANUAL

The manual content follows a project-oriented approach whereby the design is traced from preparation of the boring request through design computation of settlement, allowable footing pressure, etc., to the construction of approach embankments, pile driving operations, etc. Recommendations are presented on how to layout borings efficiently, how to minimize approach embankment settlement and eliminate the bump at the end of-the bridge, how to design the most cost-effective pile foundation, and how to transmit design information properly to construction through plans, specifications, or contact with the project engineer.

**TABLE 1**
**GEOTECHNICAL INVOLVEMENT IN PROJECT PHASES**

| Phase | Function |
|---|---|
| Planning | 1. Study existing data. (a) Topographic sheet. (b) Agricultural soil map. (c) Ground water bulletin. (d) Air photos.<br>2. Field reconnaissance with bridge engineer. (a.) Inspect nearby structures for settlement, scour, etc. (b) Assess site conditions.<br>3. Prepare terrain reconnaissance report for planning engineer. Include: (a) Anticipated soil, rock and water conditions. (b) Major problems or cost which will hinder or preclude structure construction. (c) Right-of-way required for possible special foundation treatment. (d) Beneficial shifts in alignment. |
| Alternate Design | 1. Assess structure locations with regard to major soil problems.<br>2. Provide input for Bridge Scour.<br>3. Implement subsurface program after design approval. |
| Advanced Detail Plans | 1. Review subsurface information.<br>2. Provide input for Bridge Engineer.<br>3. Submit soils investigation report to Bridge Engineer. Include: (a) Coordination with roadway construction. (b) Alternate foundation design. (c) Subsurface profile. (d) Special notes and specifications. |
| Construction | 1. Submit wave equations to Bridge Engineer. (a) Hammer approval. (b) Stress analysis. (c) Required blow count. (d) Special effects.<br>2. Attend preconstruction meeting with engineer-in-charge and pile inspector. Explain: (a) General soil profile. (b) Design basis. (c) Wave analysis. (d) Possible soil problems.<br>3. Troubleshoot soils-related problems as required.<br>4. Assist with pile load tests as required. |
| Post Construction | 1. Review actual pile results versus predicted. (a) Blow count. (b) Length. (c) Field problems. (d) Load test capacity.<br>2. Participate in court of claims action. |

The concepts presented in each chapter are concise and specifically directed at a particular operation in the foundation design process. Basic example problems are included in several sections for hands-on knowledge. Continuity between chapters is achieved by sequencing the information in the normal progression of a foundation design study. In addition , the manual contains a complete geotechnical design, in a serialized format, for a highway project involving a bridge and approach embankment over soft ground. In each phase of the fictitious project the soil concepts are developed into specific foundation designs or recommendations for that segment of the workshop design problem. The organization of the manual is presented below.

- Chapter 2 presents basic information on site investigation procedures, including terrain reconnaissance, subsurface investigation methods, standard penetration test procedures, undisturbed soil sampling, and guidelines for minimum programs in investigation of both roadway and structure sites.

- Chapter 3 discusses the basic engineering properties of the main soil groups, procedures for describing and classifying soils, and development of a soil profile.

- Chapter 4 presents effective stress principles, uses of classification test data, basic consolidation and strength testing concepts, guidelines for laboratory testing on a typical highway project, and a procedure for summarizing and choosing design values from lab tests.

- Chapter 5 and 6 present the general design procedures for stability and settlement analyses for embankments. Basic analyses are shown and explained with emphasis on practical application of analysis results to highway embankments. Remedial methods are discussed for both stability and settlement problems.

- Chapter 7 presents the foundation design procedure for shallow foundations. The analysis of both bearing capacity and settlement are discussed as well as application of results.

- Chapter 8 discusses basic concepts in the selection and design of deep foundations with emphasis on driven pile foundations. Analyses for skin friction end bearing for are covered for both cohesive and cohesionless soils. Foundation installation effects on design are discussed as well as negative skin friction and pile settlement.

- Chapter 9 provides construction control procedures for both embankments and foundations with the emphasis on control of driven pile foundations. The components of pile driving equipment, the soil properties and the use of design analysis results are related to the use of wave equation analysis in construction control. Generic information is presented on preparation of deep foundation specifications and the use of load testing.

- Chapter 10 presents a basic outline for a foundation investigation report and includes suggestions for how to incorporate geotechnical information into contract documents.

## 1.4 PRIMARY REFERENCES

A detailed list of references is provided in Chapter 11. However, certain basic references were used to develop materials for many sections in this document. In addition, FHWA has either developed or is in the process of developing detailed guidance in the topic areas covered in this document. Most of those documents are reference manuals for geotechnical courses developed for the National Highway Institute. Both the basic and detailed references are listed below. Finally, the reader is directed to the web site for the FHWA Geotechnical Group, *www.fhwa.dot.gov/bridge/geo.htm*, to obtain information on all geotechnical publications and software which have been developed by FHWA.

### 1.4.1 Basic References

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# CHAPTER 2.0
# SITE EXPLORATION FOR FOUNDATION DESIGN

To perform properly, a structure must interact favorably with the soil on which it rests. The modern foundation engineer, who often must build in areas which were considered too poor to build upon a few years past, must be well versed in the fundamentals of soil mechanics. This knowledge will be used in the design of structural foundations and earthworks to answer the following questions. Will settlements be excessive? Can the structure tolerate settlements? Will the proposed foundation type perform better than another type? Can the foundation soils safely support the imposed embankment or footing loads? Will the proposed cut or fill slopes have adequate stability?

The engineer should have adequate knowledge of the soil conditions at a site before attempting to answer these questions. By investing a few thousands of dollars into an adequate boring and testing program, costly failures or over conservative design may be prevented, resulting in design and construction savings of hundreds of thousands of dollars.

Foundation explorations should proceed through three phases

1. Initial studies and explorations to determine soil stratification and soil properties required for design.

2. Amplification, if necessary, of specific portions of the initial investigation to obtain more information both during the design phase and for preparation of contract documents.

3. Verification of anticipated foundation conditions during construction in order that changes may be made, if necessary, to either foundation design or construction procedures.

## 2.1 PREPARING FOR SITE EXPLORATION

The initial step in any highway project must include consideration of the soil or rock on which the highway embankment and structures are to be supported. The extent of the site investigation will depend on many factors, not the least of which will be the project scheduling, general subsurface conditions, and the nature of the loads to be supported. In any event, certain basic steps should be followed before a drill rig moves onto the project. The first step in the investigation is to collect and analyze all existing data.

Site exploration begins by identifying the major geologic processes which have affected the project site. Soils deposited by a particular geologic process assume characteristic topographic features, called landforms, which can be readily identified by the geotechnical engineer. A landform contains soils with generally similar engineering properties and typically extends irregularly over wide areas of a project alignment. Early identification of landforms is used to optimize the subsurface exploration program. The soil may be further described as a residual or transported soil. A residual soil has been formed at a location by the in-place decomposition of the parent material (rock). A transported soil was formed at one location and has been transported by exterior forces to a new location. Such landforms may be grouped as follows:

**Landforms**

1. TRANSPORTED SOILS

A. Aeolian (wind)

1. Sand dunes
2. Loess

B. Alluvial (water)

1. Flood plains
2. Terraces
3. Alluvial fans
4. Filled valleys
5. Coastal plains
6. Mountain outwash
7. Deltas

C. Glacial (ice)

1. Deposited by ice
   a. Moraines
   b. Till
   c. Drumlins
2. Deposited by water associated with ice
   a. Outwash
   b. Kames
   c. Eskers
   d. Lakebeds
   e. Terraces
   f. Deltas

2. RESIDUAL SOILS

A. Sedimentary

1. Flat-lying
   a. Sandstone
   b. Shale
   c. Limestone
2. Tilted
   a. Sandstone
   b. Shale
   c. Limestone
3. Interbedded

B. Igneous

1. Extrusive
   a. Basalt
   b. Volcanic cones
   c. Dikes
2. Intrusive
   a. Granite

C. Metamorphic

1. Quartzite
2. Gneiss
3. Schist
4. Serpentine
5. Slate

## 2.2 SOURCES OF EXISTING DATA

For a highway project, basic sources of geotechnical information should be reviewed to determine landform boundaries and to provide a basis for outlining the project subsurface exploration program. Those sources and functional uses are as follows:

| **Source** | **Functional Use** |
|---|---|
| 1. Topographic maps prepared by the United States Coast and Geodetic Survey (USCGS). | Current physical features shown; find landform boundaries and determine access for exploration equipment. |
| 2. County agricultural soil map sand reports prepared by the United States Department of Agriculture (USDA). | Engineering significance and boundaries of landforms shown; appraisal of general subsurface conditions. |
| 3. Air photos prepared by the United States | Detailed physical relief shown; flag major problems |

| | | |
|---|---|---|
| | Geologic Survey (USGS) or others. | such as old landslides scars, buried meander channels, or scour; provides basis for field reconnaissance. |
| 4. | Ground water resource or water supply bulletins (USGS or State agency). | Old well records or borings with general soils data shown; estimate general soils data shown; estimate required depth of explorations and pre-preliminary cost of foundations. |
| 5. | Construction plans for nearby structures (Public agency). | Foundation type and old borings shown. |
| 6. | Geology bulletins (USGS or State agency) | Type, depth and orientation of rock formations. |

The review of available data should be done prior to the field reconnaissance to establish what to look for at the site. In the eighth Rankine lecture a noted speaker stated the following truism regarding site investigation: "If you do not know what you should be looking for in a site investigation, you are not likely to find much of value."

The type of information available from USDA county soil maps is particularly useful for landforms of transported soils.

## 2.3 PRELIMINARY GEOTECHNICAL DATA (AS INTERPRETED FROM USDA SOIL MAPS)

| <u>Common Landform Type</u> | <u>General Engineering Significance for Study</u> |
|---|---|
| Sand Dune | Consider spread footings for small foundations not subject to vibratory loading. Heavy structural loads should be friction pile supported. |
| Loess | Consider spread footings for low to moderate loads. Heavy loads should be pile supported with bearing obtained below loess. Accurate ground water level determination important. |
| Flood Plain | Generally poor construction site with fine-grained soils and water problems. Potential scour area. Spread footing design below ground will probably require undercut, low foundation pressure and scour protection. Pile foundations probable. Additional shallow explorations required along footing length to determine buried meander channels. Historic high water levels should be used in design. |
| Terraces | Consider spread footings for low foundation loads. |
| Alluvial Fans | Consider spread footings for low to moderate loads except at lower elevation of alluvial fans where high water table possible. |
| Coastal Plain | Consider spread footings for moderate loads except for high water areas. Potential scour area. Soil set-up possible for friction piles. |

Moraine — Advisable to use spread footings for all foundation loads. Piles should not be used due to very difficult driving and boulders. Core all rock to 10 feet in case boulders encountered.

Glacial Till — Advisable to use spread footings for all foundation loads. Piles should not be used due to difficult driving conditions and boulders. Core all rock encountered to depth of 10 feet as large boulders may be encountered. Long-term water observations necessary to determine static water level due to soil density.

Drumlin — Suitable for spread footing design with moderate to heavy loads. Piles seldom used due to dense coarse nature of subsoil.

Outwash — Spread footing normally used to support moderate to heavy foundation loads. Piles, if required, will be short. Use large diameter sample spoon to permit representative sample to be obtained as average particle size may jam 1 3/8 inch sample spoon. Standard penetration test may be erratically high due to large particle size.

Esker — Advisable to use spread footings for all loads as soil contains much gravel and is dense. Piles not recommended. Large diameter sample spoon recommended as above for outwash.

Kame — Suitable for spread footing to support moderate to heavy foundation loads. Piles, if required, will be short. However, deposit may be associated with deep steep-sided potholes containing unsuitable material. Shallow auger sample holes recommended along footing length.

Lakebed — Only suitable for spread footing to support low loads and then settlement may be expected. Pile foundation probable and often deep. Obtain undisturbed tube samples for laboratory testing. Consider drilling with "mud" rather than casing. Long-term water observations necessary to determine static water level due to impervious soil. Potential scour area.

Delta — The use of spread footings must be carefully studied as poor soils often underlie deltaic sands and gravels. The parent material is capable of sustaining high spread footing loads. Piles may be required to penetrate delta material and poor soil. Use casing of adequate size to obtain undisturbed samples of poor soil. Potential scour area.

The area concept of site investigation allows the foundation engineer to extend the results from a limited number of explorations in a particular landform to the entire deposit. This concept is a powerful tool in reducing subsurface exploration costs and in providing the planning engineer the following useful data in the location phase:

1. Highway design — Knowledge of the landforms and of the engineering properties of the soils enables the designer to determine the most economical location for highway alignment and grade, to evaluate design problems for each type of soil deposit, and to determine sources of granular borrow.

2. Highway construction — The type and extent of problem soils to be encountered during construction may be predetermined, and construction cost more accurately estimated.

## 2.4 FIELD RECONNAISSANCE

Application of the area concept requires the use of proper subsurface exploration equipment and techniques. In particular the use of wide area exploration techniques such as remote sensing of geographical techniques can provide economical insight of general subsurface conditions in the project area. An adequate site investigation can only be accomplished under the direction of a foundation engineer who knows the general limitations of the exploration equipment as well as the general demands of the project. A site inspection, preferably with the bridge engineer, is recommended to assess foundation conditions.

The field inspection for structure related foundation problems should include:

1. Inspect any nearby structures to determine their performance with the particular foundation type utilized. If settlement is suspected, and the original structure plans are available, arrange to have the structure surveyed using the original benchmark if possible.

2. For water crossings, inspect structure footings and the stream banks up and down stream for evidence of scour. Take careful note of the streambed material. Often large boulders exposed in the stream but not encountered in the borings, are an indication of unexpected subsurface obstructions to pile installation.

3. Record the location, type, and depth of any existing structures or abandoned foundations which may infringe on the new structure.

4. Relate site conditions to proposed boring operations. Record potential problems with utilities (overhead and underground), site access, private property, or obstructions.

Figure 2 – 1 is an example of a field reconnaissance form currently used to record data pertinent to the site. Upon completion of the site inspection, the geotechnical engineer should prepare a terrain reconnaissance report assessing the general suitability of the site. The report should:

1. Flag major potential problems, which may preclude construction.

2. Recommend beneficial shifts in location.

3. Present a general discussion of expected subsurface conditions.

4. Present cost estimate for out-of-the-ordinary foundation treatments.

5. Prepare an estimate of subsurface exploration quantities, costs, and time required for completion.

This information should be transmitted to the planning unit and the bridge engineer with copies to any other involved groups. Frequent communication between drill crew, foundation engineer, bridge designer and project engineer is necessary at all stages.

## 2.5 SUBSURFACE EXPLORATION PROGRAM

The procedures employed in any subsurface exploration program are dependent on a variety of factors which vary from site to site. However, the project design objectives and the expected site soil conditions have a major influence on the subsurface explorations. Highway projects necessarily involve both

BRIDGE FOUNDATION INVESTIGATION
FIELD RECONNAISSANCE REPORT
STATE HIGHWAY DEPARTMENT OF ________

Project No:__________ County:__________ Sta. No.: ___________
Reported by: ___________________________________

1. STAKING OF LINE
___ Well Staked
___ Poorly Staked (we can work)
___ Request Division to Restake

2. BENCH MARKS
In Place: Yes ___ No ___
Distance from Bridge - Ft. ___

3. PROPERTY OWNERS
Granted Permission: Yes ___ No ___
Remarks on Back ___

4. UTILITIES
Will Drillers Encounter Underground or Overhead Utilities? Yes ___ No ___
Maybe ___ At which Holes? ______
What Type? __________________
Who to see for Definite Location ___
__________________ Feet _____

5. GEOLOGIC FORMATION

6. SURFACE SOILS
Sand ___ Clay ___ Sandy Clay ___
Muck ___ Silt ___ Other ___

7. General Site Description
Topography
Level ___ Rolling ___ Hillside ___
Valley ___ Swamp ___ Gullied ___
Groundcover
Cleared ___ Farmed ___ Buildings ___
Heavy Woods ___ Light Woods ___
Other ______________________
Remark on Back ________________

8. BRIDGE SITE
Replacing ____________________
Relocation ___________________
Check Appropriate Equipment
___ Truck Mounted Drill Rig
___ Track Mounted Drill Rig
___ Failing 1500
___ Truck Mounted Skid Rig
___ Skid Rig

Bridge Site – Cont'd
___ Rock Coring Rig
___ Wash Boring Equipment
___ Water Wagon
___ Pump
___ Hose ___ Feet
Cut Section – Feet ___
Fill Section – Feet ___
If Stream Crossing:
Will Pontoons be Necessary? ____
Can Pontoons be Placed in Water Easily?
_________________
Can Cable be Stretched Across Stream? ____
How Long? ____
Is Out board Motorboat Necessary? ____
Current: Swift ___ Moderate ___ Slow __
Describe Streambanks score.
If Present Bridge Nearby:
Type of Foundation ________________
Any Problem Evident in Old Bridge Including Scour ______(describe on back)
Is Water Nearby for Wet Drilling – Feet ___
Are Abandoned Foundation in Proposed Alignment? ______________________

9. GROUND WATER TABLE
Close to Surface – Feet _____
Nearby Wells – Depth – Feet ________
Intermediate Depth – Feet ___

10. ROCK
Boulders Over Area? Yes ___ No ___
Definite Outcrop? Yes ___ No ___
(show sketch on back) What kind? ___

11. SPECIAL EQUIPMENT NECESSARY

12. REMARKS ON ACCESS – Describe any problems on Access

13. DEBRIS AND SANITARY DUMPS
Stations _____________________
Remarks ____________________

Reference: 1978 AASHTO Foundation Investigation Manual

Figure 2 – 1: Typical Field Reconnaissance Form

embankment and structure foundations. Typical boring programs for highways on new location are established such that basic information is first gathered along the entire highway alignment and subsequent detailed borings are taken as required at structures or in problem embankment areas disclosed by the initial basic program. Subsurface explorations for widening or improvements of existing highways generally are done in one stage as location is predetermined.

## 2.6 GENERAL HIGHWAY EXPLORATIONS

Embankments are less sensitive than structures to variations in subsurface conditions. Embankment loads are spread over a wide area while structure loads are concentrated. Designers of highways in cut sections are less concerned with deep exploration of subsurface conditions than defining the properties of the soil or rock on which the subgrade materials will be placed. The subsurface exploration program for embankments or cuts must be necessarily widely spaced as the major portion of a highway alignment is one or the other. This section of the manual will deal primarily with approach embankments. Highway embankment and cut explorations are done using the same procedures, but the spacing and depth of borings vary. FHWA Demonstration Project 12, “Soils Exploration and Testing”, suggests that general spacings for embankment or cut borings be 200 to 400 feet with at least one boring in each landform. Highway embankment borings are generally extended to a depth equal to either twice the embankment height or based on landform type and geologic conditions. Cut borings are extended at least 15 feet beyond the anticipated depth of cut at the ditch line. Soft ground conditions at the location of highway embankments or cuts may require additional borings or special testing using methods to be described below.

Approach embankments require more detailed exploration than other highway embankment areas as stability and settlement values must be established before structure foundation design. Typically, test borings (drill holes) are taken for the approach embankment and located at proposed abutment locations to serve a dual function. The depth of the boring will usually be determined by criteria established for the structure design which is described in the following section. In all cases, a boring will extend a distance into competent soil or rock. Additional shallow explorations (auger holes) are commonly taken at approach embankment locations to explore the depth of any suspected unsuitable surface soils or determine topsoil thickness. Additional detailed guidance is available in FHWA-ED-88-053, "Checklist and Guidelines for Review of Geotechnical Reports". Various types of commonly used explorations are shown in Table 2 – 1. The objectives of either deep or shallow borings is to obtain information and samples necessary to define soil and rock subsurface conditions as follows:

1. Stratigraphy.
    a. Physical description and extent of each stratum.
    b. Thickness and elevation of various locations of top and bottom of each stratum.

2. For cohesive soils (each stratum).
    a. Natural moisture contents.
    b. Atterberg limits.
    c. Presence of organic materials.
    d. Evidence of desiccation or previous soil disturbance, shearing, or slickensides.
    e. Swelling characteristics.
    f. Shear strength
    g. Compressibility

3. For granular soils (each stratum).
    a. In-situ density (average and range) typically determined from Standard Penetration Tests or Cone Tests.

b. Grain-size distributions (gradation).
c. Presence of organic materials.

4. Ground water (for each aquifer if more than one is present).
   a. Piezometric surface over site area, existing, past, and probable range in future (observe at several times).
   b. Perched water table.

5. Bedrock
   a. Depth over entire site.
   b. Type of rock.
   c. Extent and character of weathering.
   d. Joints, including distribution, spacing, whether open or closed, and joint infilling.
   e. Faults.
   f. Solution effects in limestone or other soluble rocks.
   g. Core recovery and soundness (RQD).

Numerous tools exist for sampling soils including the Pitcher sampler, the Dennison sampler, and drive samplers or augers. When soft ground is encountered, field (in situ) testing and/or undisturbed sample explorations should be done. The use and limitations of undisturbed sampling equipment and in situ testing are respectively shown in Tables 2 – 1 and 2 – 2.

**TABLE 2 – 1**
**SUBSURFACE EXPLORATION – EXPLORATORY BORING METHODS**

| Method | Use | Limitations |
|---|---|---|
| Auger Boring ASTM D – 1452 | Obtain samples and identify changes in soil texture above water table. Locate groundwater. | Grinds soft particles – stopped by rocks, etc. |
| Test Boring ASTM D – 1586 | Obtain disturbed split spoon samples for soil classification. Identify texture and structures; estimate density or consistency in soil or soft rock using SPT (N). | Poor results in gravel hard seams. |
| Thin Wall Tube ASTM D – 1587 | Obtain 2” to 3-3/8” diameter undisturbed samples of soft-firm clays and silts for later lab testing. | Cutting edge wrinkled in gravel. Samples lost in very soft clays and silts below water table. |
| Stationary Piston Sampler | Obtain undisturbed 2” to 3-3/8” diameter samples in very soft clays. Piston set initially at top of tube. After press is completed, any downward movement of the sample creates a partial vacuum which holds the sample in the tube. | Cutting edge wrinkled in gravel. |
| Pits, Trenches | Visual examination of shallow soil deposits and man made fill above water table. Undisturbed block samples may be extracted. | Caving of walls, ground water. |

When additional undisturbed sample borings are taken, the undisturbed samples are sent to a soils laboratory for testing. Drilling personnel should exercise great care in extracting, handling, and transporting these samples to avoid disturbing the natural soil structure. Tubes should only be pressed, not driven with a hammer. The length of press should be 4 to 6 inches less than the tube length (DO NOT OVERPRESS). A plug composed of a mixture of bees wax and paraffin should be poured to seal the tube against moisture loss. The void at the upper tube end should be filled with sawdust and then both ends capped and taped before transport. The most common sources of disturbance are rough, careless handling of the tube (such as dropping the tube samples in the back of a truck and driving 50 miles over a bumpy road), or temperature extremes (leaving the tube sample outside in below zero weather or storing in front of a furnace). Proper storage and transport should be done with the tube upright and encased in an insulated box partially filled with sawdust or styrofoam to act as a cushion. Each tube should be physically separated from adjacent tubes like bottles in a case. An alternate method to ease transportation and storage problem is to extrude the soil from the tube in the field. These samples should be carefully sectioned in 6 to 8 inch lengths, wrapped in wax paper and sealed in a cardboard container (such as ice cream cartons) using liquid paraffin.

**TABLE 2 – 2**
**SUBSURFACE EXPLORATION IN SITU TESTS**

| Type of Test | Best Suited For | Not Applicable | Properties That Can be Determined | Remarks |
|---|---|---|---|---|
| A. Routine Accepted Tests* | | | | |
| Standard Penetration Test (SPT) AASHTO T – 206 | Sand, Clay | Gravel | Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification. Estimate of Friction Angle, φ. | Test best suited for sands. Estimated of clay shear strength are crude & should not be used for design. |
| Dynamic Cone Test | Sand, Gravel | Clay | Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification | FHWA TS-78-209 |
| Static Cone Test ASTM D3441 | Sand, Clay | -- | Continuous evaluation of density and strength of sands and gravels. Evaluation of pore pressure and undrained shear strength in clays. | Use piezo-cone for pore pressure data. Tests in clay are reliable only when used in conjunction with vane tests. FHWA SA-91-043 |
| Vane Shear Test AASHTO T-223 | Clay | Silt, Sand, Gravel | Undrained shear strength, $C_u$. | Test should be used with care particularly in fissured, varved and highly plastic clays. Extract sample of material tested. |
| Permeability Test | Sand, Gravel | Clay | Evaluation of Coefficient of Permeability | Variable head test in boreholes have limited accuracy. Results reliable to one order to magnitude are obtained only from long-term, large scale pumping tests. |
| B. Recently Developed Tests | | | | |
| Pressure-meter Test ASTM D4719 | Soft rock, Sand, Clay | -- | Ultimate bearing capacity and compressibility | Requires highly skilled field personnel (FHWA IP-89-008) |
| Borehole Shear Devices | Sand, Soft Clay | Stiff Clay | Shear Strength | (See FHWA RD-81-1109) |
| Dilatometer | Sand, Clay | -- | Average grain size, Horizontal stress, soil stiffness. | Introduce in USA in 1981 (see ASCE Geot. Journal 3/80 and FHWA SA-91-044). |

*(Table based on Canadian Foundation Engineering Manual)

## 2.7 PROCEDURE FOR SHELBY TUBE SAMPLING

The following procedure was prepared by the New York DOT and modified by the Oregon DOT for successful undisturbed tube sampling. This procedure should be included in agency standards and specifications for undisturbed sampling performed by contract drillers.

**General Purpose:**

Thin wall tube samples (Shelby tube) are taken to obtain undisturbed samples for laboratory testing to obtain the strength and settlement properties of fine-grained soils containing silt and clay and in some cases organic material. It is extremely important that the samples be pressed and transported with a minimum amount of disturbance. Poor sampling practices, exposure to extreme temperatures and careless handling of samples causes misleading test results that could result in uneconomical designs.

Sampling procedures will be the same regardless of size of Shelby tube. A serious attempt should be made to minimize the length of time between sample procurement and delivery to the central lab. If at all possible, samples should be shipped to the lab the day following procurement of the sample in the field. Careful handing is of utmost importance for geotechnical design units to have reliable information. This careful handling begins with receiving the Shelby tubes from the manufacturer.

1. Receiving Shelby tubes:

   Shelby tubes are received from the manufacturer packed several to the box. Upon receipt, the tubes should be removed from the container immediately and plastic caps put on both ends to prevent damage to these ends from subsequent handling. The tubes should then be stored in an area by themselves, out of the weather, at room temperature, and kept in a horizontal position. Tubes sent from the manufacturer generally have a light film of oil on all surfaces. If not, then a thin coat of oil (such as WD-40) should be applied. This will prevent rust from forming which could affect the sample quality.

2. Transportation of tubes to and from the field:

   Shelby tubes should be placed in a Shelby tube rack as provided by the central lab whenever they are transported to and from the field, whether they are full or empty. Figure 2-2 shows an example of a Shelby tube rack. This rack is to be maintained in a vertical position at all times and all full tubes will remain in the rack until removed by the lab technicians. Transportation to and from the field should be done with the tubes kept inside a vehicle where room temperature can be maintained at all times if possible.

3. Cleaning out the hole:

   The hole should be cleaned out thoroughly before sampling. Clean-out should be done with a clean-out jet type auger for the last 6". In very soft soils, only side discharge auger bits should be used. Bottom discharge bits may cause jet holes in the center of the sample.

   Whenever possible, hollow stem augers should be used for minimum soil disturbance below the bit. At shallow depths hand auger equipment can be utilized to advance the hole.

4. Rate of press:

Shelby tubes should be advanced by a smooth continuous operation. A continuous fast press may be used taking less than 5 seconds. Under no circumstances should a 30" tube be advanced more than 24" to allow for loose material in the hole. For soft soils, wait 5 to 15 minutes before rotating the sampler to shear the end of the sample. For firm soils, a waiting period may not be required. A 360-degree rotation of the drill rod and sample is mandatory to shear the soil at the tube tip elevation.

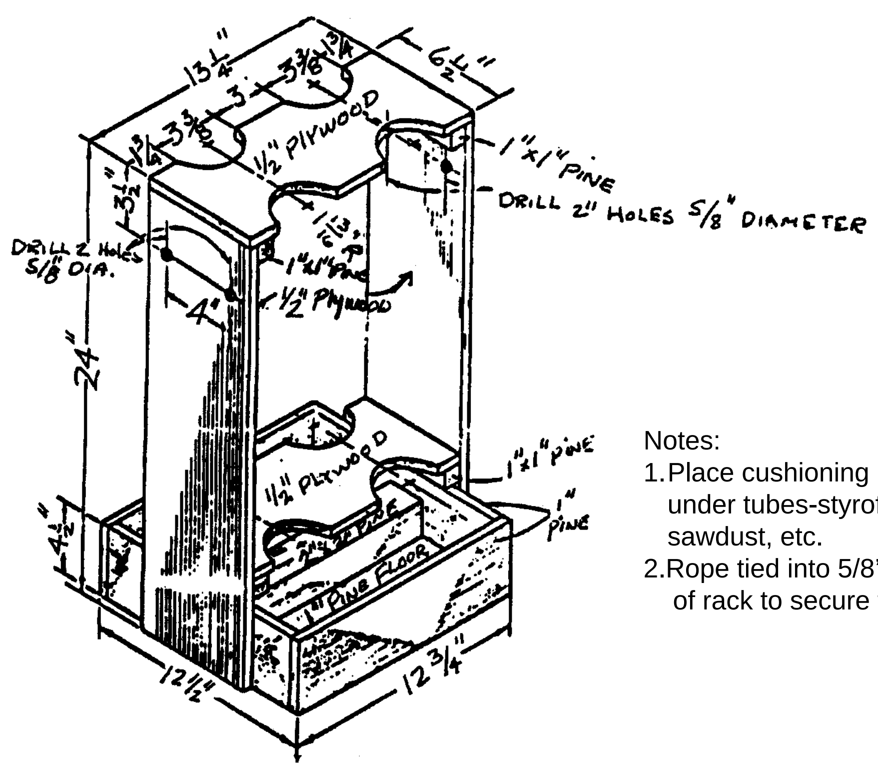
5. Recovery of tube:

   The Shelby tube, after shearing, should be recovered from the hole in much the same manner it was pressed into the hole--with a smooth continuous motion with no jerking.

6. Tube preparation (Figure 2-3):

   Preparation of the Shelby tube for transport to the lab is very critical and meticulous care must be taken to the fine details of this part of the operation.

   The thin wall tube is carefully removed from the sampler head. The tip or bottom of the sampler is scraped smooth so there is no disturbance of the material inside of the tube. The tube is then placed into a holder with the same orientation to vertical as when the sample was taken. A mixture of 50% paraffin and 50% beeswax is melted ahead of time for use in sealing the tube. The top of the tube is cleared of loose material so that the wax is poured on a reasonably smooth soil surface. A wax plug approximately 1/4" thick is poured into the tube by using a funnel in order to keep wax off the sides of the tube. This plug is allowed to set up and then another 1/4" thick layer is poured on top of that. This reduces the shrinkage factor.




Notes:
1. Place cushioning material under tubes-styrofoam or sawdust, etc.
2. Rope tied into 5/8'' holes at top of rack to secure top of tubes

Figure 2-2: Shelby Tube Rack

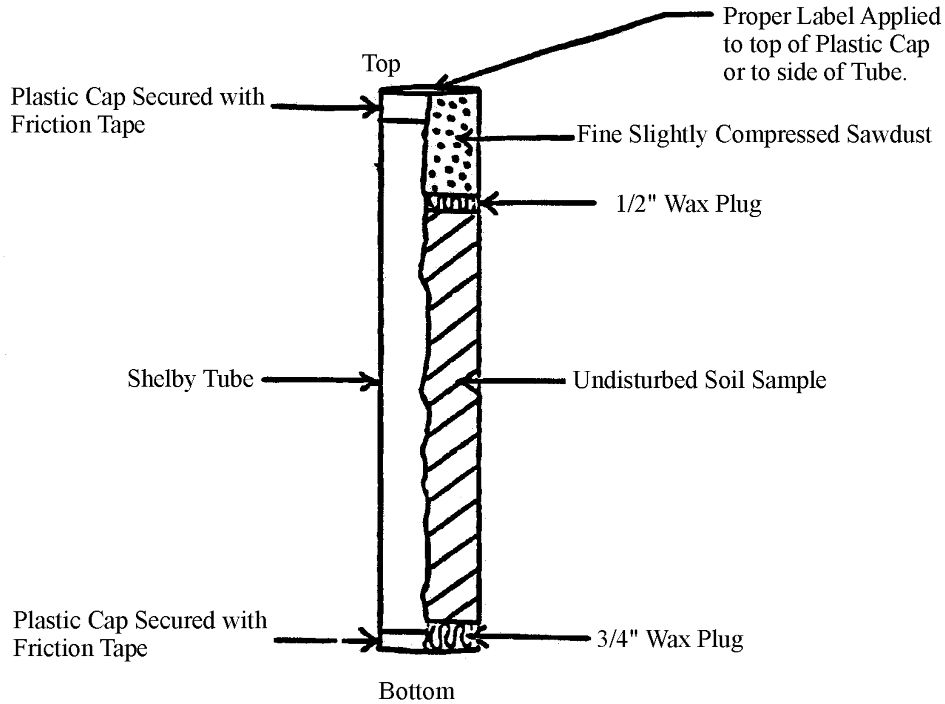



Figure 2-3: Shelby Tube Preparation

After the wax sets up, the remainder of the open tube is filled with fine, lightly compressed sawdust. The end of the tube is then capped with a plastic cap and secured with friction tape. Print the hole number and sample number on top cap with magic marker. This is a precaution against label being lost.

The tube is then turned upside down and the bottom end is prepared for sealing. Approximately 3/4" of material is scraped out of the end of the tube, leaving a smooth surface. The material removed is put into a moisture sample jar or can for further testing. An approximately 3/8" thick plug of wax is poured over the soil. When this plug sets up, the remainder of the tube is filled with wax. When this wax sets up, a plastic cap is fitted over the end and secured with friction tape.

An identification label should be taped to the side of the tube or onto the top cap. The label should include the project identification number, sample number, project name, hole number, station, offset, depth of sample, depth pressed, recovery, and any remarks by the driller.

The tube is then placed into its rack in proper vertical orientation (cutting the edge) and stored inside at room temperature until shipment.

Sealing of samples taken in wet weather should be done under an overhead shelter, as the rain or moisture might affect the quality of the samples taken.

7. Tube shipment to lab:

When all samples have been taken or when the tube rack is full, care must be taken to make sure that the samples are not exposed to extreme cold (freezing) or heat. The best method for this is to completely fill the tube rack with fine sawdust prior to shipping, and to ship in a vehicle where room temperature can be maintained.

## 2.8 STANDARD PENETRATION TEST (SPT)

Probably the most widely used field test in the United States is the Standard Penetration Test (SPT). This test has been standardized in both AASHTO T-206 and ASTM D-1586. SPT testing is recommended for all drill holes taken on highway projects due to the simplicity and economy of the test and the usefulness of the data obtained. In this test, a measure of soil density and a soil sample are obtained in the following manner. After the boring is cleaned out, the standard split-spoon sampler is attached to a set of drill rods and lowered to the bottom of the hole. Attached to the upper end of the drill rod is a 140-pound hammer which can be hand operated. The test consists of driving the split-spoon sampler 18 inches with the 140-pound hammer falling through a drop of 30 inches. The first 6-inch increment is referred to as the seating load. The sum of the next two 6-inch increments is known as the Standard Penetration Value (N). The soil sample obtained is disturbed, but can be used for visual classification. The sample is normally tightly sealed in a jar and sent to the laboratory where routine tests such as natural water content, gradation analyses, and Atterberg Limits can be conducted.

The N-values of this test are an indication of the density of cohesionless soils and the consistency of cohesive soil. General N-value ranges versus density and consistency are shown below in Table 2-3. It is emphasized that for gravels and clays these are rather unreliable and should only serve as general estimates.

**TABLE 2 – 3**
**SOIL PROPERTIES CORRELATED WITH STANDARD PENETRATION TEST VALUES***

| Sands (Reliable) | | Clays (Rather Unreliable) | |
|---|---|---|---|
| Number of Blows per ft, N | Relative Density | Number of Blows per ft, N | Consistency |
| 0-4 | Very loose | Below 2 | Very soft |
| 5-10 | Loose | 2-4 | Soft |
| 11-30 | Medium Dense | 5-8 | Medium |
| 31-50 | Dense | 9-15 | Stiff |
| Over 50 | Very dense | 16-30 | Very stiff |
| | | Over 30 | Hard |

*Measured with 1-3/8" I.D., 2" O.D. sampler driven by 140# hammer falling 30".
*Sections 6.3.2 and 7.2.1 contain additional information on the uses of SPT values to estimate engineering properties.

### 2.8.1SPT Test Errors

Although the procedures have been standardized for conducting the SPT test, several errors can creep into the test. The most common errors are:

1. Effect of overburden pressure. Soils of the same density will give smaller counts near the ground surface.

2. Variations in the 30-inch free fall of the drive weight, since this is often done by eye on older equipment using a rope wrapped around a power takeoff (cathead) from the drill motor. Newer automatic hammer equipment does this automatically.

3. Interference with the free fall of the drive weight by the guides or the hoist rope. New equipment eliminates rope interference.

4. Use of a drive shoe that is badly damaged or worn from too many drivings to "refusal" (blow count exceeding 100).

5. Failure to properly seat the sampler on undisturbed material in the bottom of the boring.

6. Inadequate cleaning of loosened material from the bottom of the boring.

7. Failure to maintain sufficient hydrostatic pressure in the borehole during drilling. Unbalanced hydrostatic pressures between the borehole drill water and the ground water table can cause the test zone to become "quick." This can happen when using the continuous-flight auger with the end plugged and maintaining a water level in the hollow stem below that in the hole.

8. SPT results may not be dependable in gravel. Since the split-spoon inside diameter is 1-3/8 inches, gravel sizes larger than 1-3/8 inches will not enter the spoon. Therefore, soil descriptions may not reflect actual gravel content of the deposit. Also, gravel pieces may plug the end of the spoon and cause the SPT blow count to be erroneously high.

9. Samples retrieved from dilatant soils (fine sands, sandy silts) which exhibit unusually high blow count should be examined in the field to determine if the sampler drive shoe plugged. Look for poor sample recovery as an indication of plugging.

10. Careless work on the part of the drill crew.

THE USE OF RELIABLE QUALIFIED DRILLERS CANNOT BE OVEREMPHASIZED. AGENCIES WHICH MAINTAIN THEIR OWN DRILLING PERSONNEL AND EQUIPMENT ACHIEVE MUCH MORE RELIABLE, CONSISTENT RESULTS THAN THOSE WHO ROUTINELY LET BORING CONTRACTS TO THE LOW BIDDER.

Studies show that soil type, density, and overburden pressure are the most significant factors affecting "N" (assuming good workmanship and equipment).

Regardless of the impressive list of shortcomings, the SPT is not likely to be abandoned for several reasons:

1. The test is very economical in terms of cost per unit of information.
2. The test results in recovery of soil samples, which can be tested for index properties and visually examined.
3. Long service life of the enormous amount of equipment in use.
4. The accumulation of a large SPT database which is continually expanding.
5. The fact that other methods can be readily used to supplement the SPT when the borings indicate more refinement in sample/data collection.

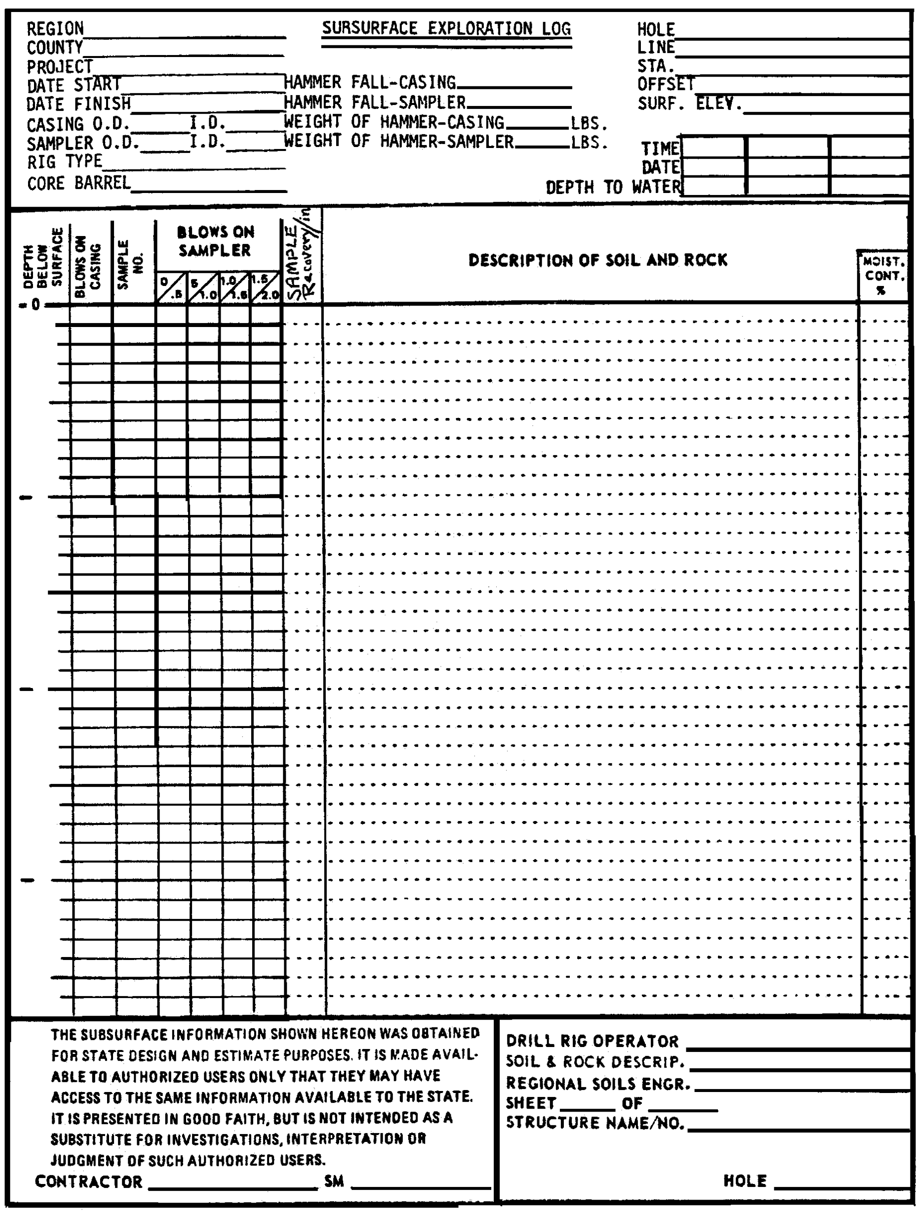
## 2.9 FIELD BORING LOG

The importance of good logging and field notes cannot be overemphasized. The logger must realize that a good field description must be recorded. The field-boring log is the major portion of the factual data used in the analysis of foundation conditions.

The log is a record which should contain all of the information obtained from a boring whether or not it may seem important at the time of drilling. It is important to record the maximum amount of accurate information. This record is the "field" boring log, as opposed to the "finished" boring log used in the preparation of the final report made to the designer. The finished log is drawn from the data given in the field log supplemented by the results of lab visual identification of samples and lab classification tests. A typical boring log form which can be used for recording both field and finished data is shown on Figure 2-4.

The person who actually logs the field information will vary from organization to organization. Some will have an engineering geologist, or trained technician accompany the drill crew, while others may train the drill crew foreman to log the borehole. In order to obtain the maximum amount of accurate data, the logger should work closely with the driller and be alert for changes in materials and operations while drilling.

SUBSURFACE EXPLORATION LOG



REGION ______
COUNTY ______
PROJECT ______
DATE START ______
DATE FINISH ______
CASING O.D. ______ I.D. ______
SAMPLER O.D. ______ I.D. ______
RIG TYPE ______
CORE BARREL ______

SURSURFACE EXPLORATION LOG

HAMMER FALL-CASING ______
HAMMER FALL-SAMPLER ______
WEIGHT OF HAMMER-CASING ______ LBS.
WEIGHT OF HAMMER-SAMPLER ______ LBS.

HOLE ______
LINE ______
STA. ______
OFFSET ______
SURF. ELEV. ______

| | | | |
|---|---|---|---|
| TIME | | | |
| DATE | | | |
| DEPTH TO WATER | | | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | SAMPLE Recovery/in | DESCRIPTION OF SOIL AND ROCK | MOIST. CONT. % |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | |

THE SUBSURFACE INFORMATION SHOWN HEREON WAS OBTAINED FOR STATE DESIGN AND ESTIMATE PURPOSES. IT IS MADE AVAILABLE TO AUTHORIZED USERS ONLY THAT THEY MAY HAVE ACCESS TO THE SAME INFORMATION AVAILABLE TO THE STATE. IT IS PRESENTED IN GOOD FAITH, BUT IS NOT INTENDED AS A SUBSTITUTE FOR INVESTIGATIONS, INTERPRETATION OR JUDGMENT OF SUCH AUTHORIZED USERS.

CONTRACTOR ______ SM ______

DRILL RIG OPERATOR ______
SOIL & ROCK DESCRIP. ______
REGIONAL SOILS ENGR. ______
SHEET ______ OF ______
STRUCTURE NAME/NO. ______
HOLE ______

Figure 2 – 4: Subsurface Exploration Log

Duties of the Logger:

Generally, the logger should be responsible for recording the following information on the field boring log:

1. General description of each rock and soil stratum, and the depth to the top and bottom of each stratum.

2. The depth at which each sample is taken, the type of sample taken, its number, and any loss of samples taken during extraction from the hole.

3. The depths at which field tests are made and the results of the test.

4. Information generally required by the log format, such as:

   - Boring number and location.
   - Date of start and finish of the hole.
   - Name of driller (and of logger, if applicable).
   - Elevation at top of hole.
   - Depth of hole and reason for termination.
   - Diameter of any casing used.
   - Size of hammer and free fall used on casing (if driven).
   - Blows per foot to advance casing (if driven).
   - Description and size of sampler.
   - Size of drive hammer and free fall used on sampler in dynamic field tests.
   - Blow count for each 6 inches to drive sampler. (Sampler should be driven three 6-inch increments or to 100 blows).
   - Type of drilling machine used.
   - Type and size of core barrel used.
   - Length of time to drill each core run or foot of core run.
   - Length of each core run and amount of core per run.
   - Recovery of sample in inches and RQD of rock core.
   - Project identification.

5. Notes regarding any other pertinent information and remarks on miscellaneous conditions encountered, such as:

   - Depth of observed groundwater, elapsed time from completion of drilling, conditions under which observations were made, and comparison with the elevation noted during reconnaissance (if any).
   - Artesian water pressure.
   - Obstructions encountered.
   - Difficulties in drilling (caving, coring boulders, surging or rise of sands in casing, caverns, etc.).
   - Loss of circulating water and addition of extra drilling water.
   - Drilling mud and casing as needed and why.
   - Odor of recovered sample.
   - Sampler plugged.
   - Poor recovery.

6. Any other information the collection of which may be required by highway agency policy.

## 2.10 GUIDELINES FOR MINIMUM SUBSURFACE EXPLORATION PROGRAM

In regard to the scope of the subsurface program for a structure, one must carefully consider the small cost of a boring in relation to the foundation cost. A 2½ - inch diameter drill hole will cost less than one 12-inch diameter pile. Yet the knowledge gained from that boring will permit proper design techniques to be used which may allow elimination of all piles for that structure. Without adequate boring data, the foundation design engineer cannot utilize his technique or experience and must rely on extremely conservative designs with high safety factors.

Planning a soils or foundation exploration program should include determining the depth and location of borings, test pits, or other procedures to be used and establishing the methods of soil sampling and testing to be employed. Usually, the extent of the work is established as it progresses, unless knowledge of foundation conditions is available from geological studies, earlier investigations, or records of existing structures. The number, depth, spacing, and character of tests to be made in any individual exploration program are so dependent upon site conditions, type of structure, and its requirements, that no rigid rules may be established. However, certain general principles for the guidance of those charged with the investigation can be outlined.

The following program will produce the minimum foundation data for a typical structure site. Soft ground conditions may require undisturbed sample explorations or in situ testing as previously mentioned.

1. Progress one minimum 2½ - inch diameter drill hole at each pier or abutment, and at the end of any wingwall which measures over 30 feet in length. The hole pattern should be staggered at the opposite ends of adjacent footings. Piers or abutments over 100 feet in length require one 2½ - inch drill hole at the extremities of each element. The drill holes may be advanced with casing, drilling mud, or continuous flight augers. For proposed spread footing design on sloping rock surfaces, additional borings and probe holes may be required.

2. Estimate the boring depth from existing data obtained during the terrain reconnaissance phases or, less preferred, from requested boring resistance data such as: "The borings for structure foundations shall be terminated when a minimum resistance criteria of 20 blows per foot on the sample spoon has been achieved for 20 feet of drilling," or "the boring shall extend 10 feet into rock having an average recovery of 50 percent or greater." The minimum resistance criteria may be modified depending on the deep foundation capacity anticipated at the site.

3. Obtain standard split spoon samples at 5-foot intervals or at changes in material. Continuous spoon samples are recommended for the top 15 feet where footings may be placed on natural soil. These spoon samples are "disturbed" samples generally not suited for laboratory determination of strength or consolidation parameters. Undisturbed Shelby tube samples should be obtained at 5-foot intervals in at least one boring in cohesive soils. For cohesive deposits greater than 30 feet in depth, tube sample interval can be increased to 10 feet. In soft clay deposits in situ vane shear strength tests are recommended at 5-10 foot intervals.

4. Record the standard penetration test data for each drill hole in accordance with ASTM D-1586. The SPT test is the most economical method presently available of procuring useful data regardless of the often cited frailties of the test.

5. Instruct the drilling crew to perform a rough visual analysis of the soil samples and record all pertinent data on a standard drill log form. The disturbed spoon samples must be carefully sealed in plastic bags, placed in jars, and sent to the laboratory for analysis. Undisturbed tube samples must be sealed and

stored upright in a shock proof, insulated container normally constructed from plywood and filled with cushioning material.

6. Observe the water level in each boring and record the depth below top of hole and the date of the reading on the drill log for:

   a. Water seepage or artesian pressure encountered during drilling. Artesian pressure may be measured by extending drill casing above the ground until flow stops. Report the pressure as the number of feet of head above ground.

   b. Water level at the end of each day and at completion of boring.

   c. Water level 24 hours (minimum) after hole completion. Long term readings may require installation of a perforated plastic tube before abandoning the hole.

   A false indication of water level may be obtained when water is used in drilling and adequate time is not permitted after hole completion for the water level to stabilize. In low permeability soils, such as clays, more than one week may be required to obtain accurate readings.

7. Designate a unique identification number for each drill hole to prevent duplication during later exploration phases. Much confusion has resulted on projects where exploration numbering was done by only single numbers. It was not unusual to have several drill holes numbered DH-1 on the same project. A suggested method to avoid duplication is to designate that all bridge holes begin with the letter "B", followed by the initials of the highway or river being crossed and finally a sequential number, i.e., the first hole for the Apple Freeway structure would be designated DH-BAF-1.

The reasons for obtaining this minimum data are clear; the engineer must have adequate data to determine the soil type and relative compactness, and the position of the static water level. Methods such as driving open-end rod without obtaining soil samples or water level readings taken after the last soil sample was removed must be discouraged. Good communication between the driller and the foundation engineer is essential during all phases of the subsurface investigation program.

## 2.11 APPLE FREEWAY DESIGN EXAMPLE – SITE EXPLORATION

In each chapter various pertinent aspects of the design process as outlined in Section 1.3 are demonstrated through a fictitious bridge project named "Apple Freeway Project." The plan and cross section of the fictitious bridge and approaches are shown in Figure 2 – 5. The following design example presents the process of planning a site exploration program for Apple Freeway Project.

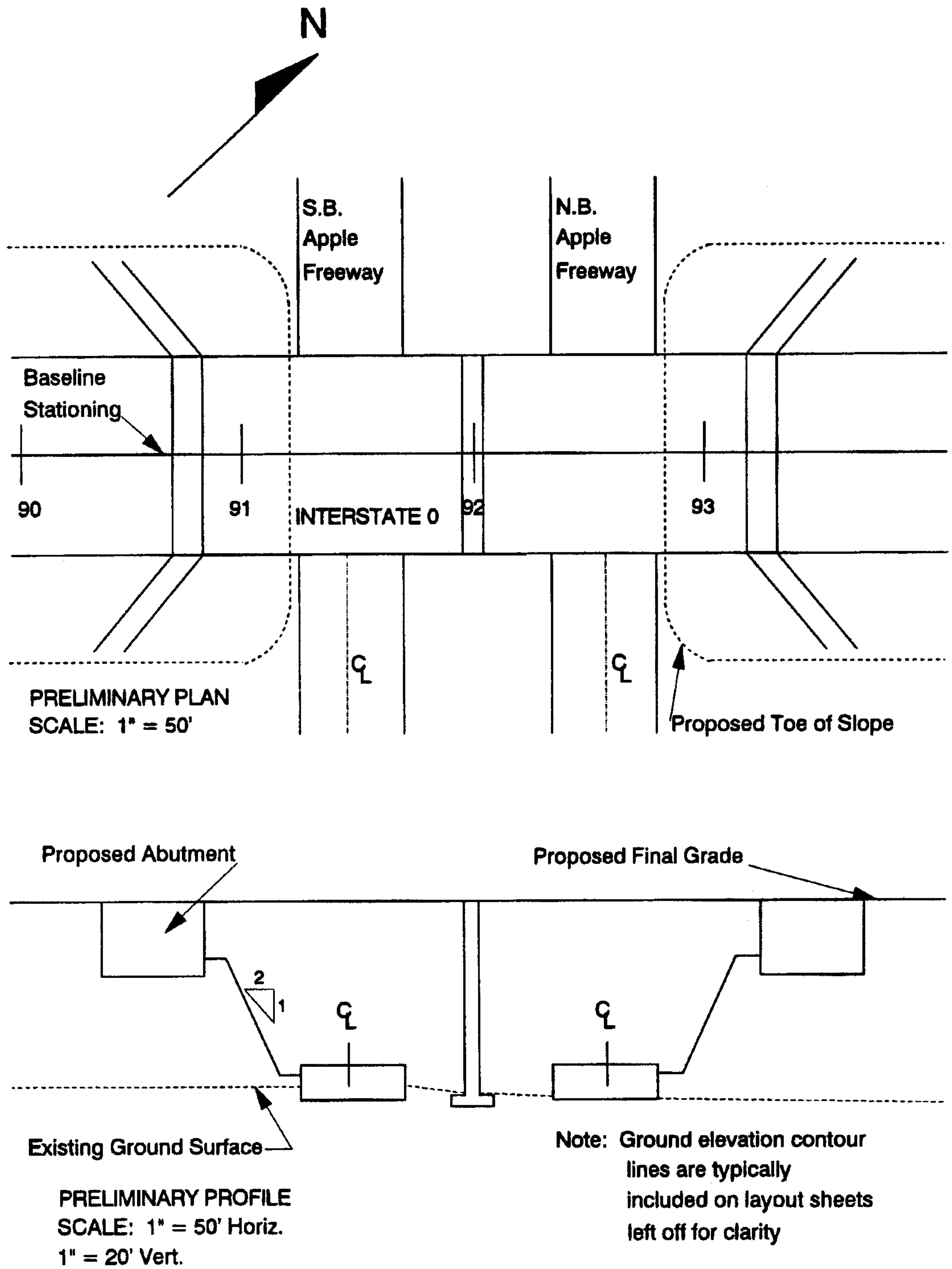



Figure 2 – 5: Apple Freeway Plan and Section

## Site Exploration

Terrain Reconnaissance
Site Inspection
Subsurface Borings

| | |
|---|---|
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | Po Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

Apple Freeway Design Example – Site Exploration
Exhibit A

## LAYOUT OF SUBSURFACE EXPLORATION PROGRAM

**Given:** Soil map showed structure to be located in a delta landform. Field inspection showed wet area with cattails in vicinity of East abutment.

**Required:** Plan subsurface exploration program and prepare boring request.

**Solution:**

**Step 1:** **Identify boring types required and location established (see exhibit B).**

- Disturbed SPT sample boring at each abutment and intermediate support
- Hand Auger holes in wet area within East approach fill limits

**Step 2:** **Establish criteria for determining boring depth.**

- SPT holes to depth where the minimum N average equals 20 for 20′ depth or 10′ into bedrock whichever depth is less.
- Hand auger holes to a maximum depth of 10′ or at least 3′ below bottom of unstable soils (soft and/or organic soils) whichever depth is less.

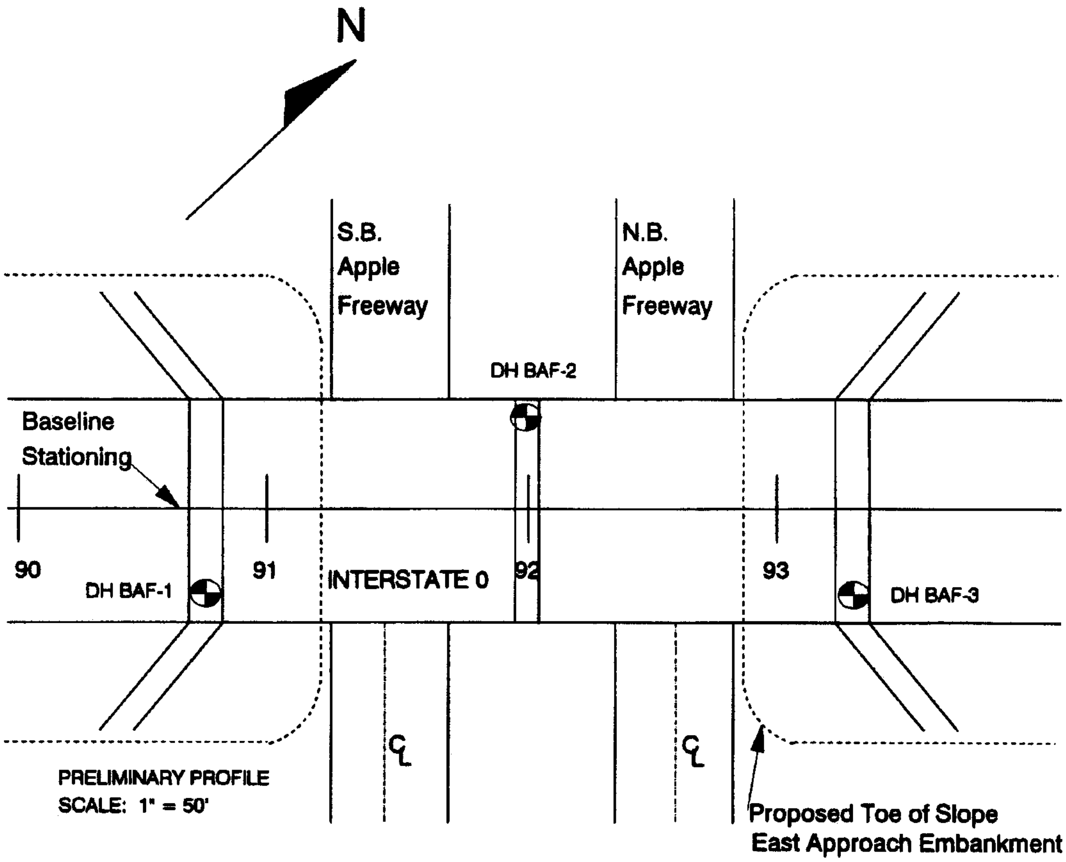
**Step 3:** **Establish sampling criteria**

- East and West abutments: Disturbed SPT every 5′.
- Pier footing: Continuous SPT samples to depth of 15′, then 5′ intervals.
- Wet area: obtain representative samples in each auger hole.

**Step 4:** **Identify other important consideration.**

- Since area is a delta landform, granular deposits overlying clay may be encountered. If so, an undisturbed drill hole (UDH) will be required. The location, depth, and sampling details will be selected based on the results of the three SPT boring. Notify the drillers of possibility of UDH and vane shear so necessary equipment can be taken to site. Long-term water level reading should be taken in one hole.

**Step 5:** **Prepare boring request (see exhibit C).**




Apple Freeway Design Example – Site Exploration
Exhibit B - Proposed Site Explorations

WORKSHOP DESIGN EXAMPLE
BORING REQUEST

March 1, 1992

Subject: Request for Subsurface Investigation
Interstate Structure over the Apple Freeway

From: Foundation Engineer

To: Regional Office

In accordance with project authorization from the Chief Engineer dated January 16, 1992, a subsurface investigation program has been prepared for the subject structure. We request that your office progress a 2½ - inch diameter cased drill hole at each of the following locations:

| Hole No. | Baseline Station | Offset (ft) |
|---|---|---|
| DH-BAF-1 | 90 + 77 | 50' Rt |
| DH-BAF-2 | 92 + 00 | 50' Lt |
| DH-BAF-3 | 93 + 27 | 50' Rt |

The locations may be field adjusted along the footing line shown on the attached drawing if necessary.

Each boring shall extend to a depth where the blow count per foot on the sample spoon has exceeded 20 for a 20' depth. If rock is encountered above this depth, 10 feet of rock core shall be extracted. Spoon samples shall be taken at intervals of 5-feet except for the top 15 feet of BAF-2 where continuous spoon samples are required. On completion of BAF-2 a perforated plastic pipe should be inserted before extracting the casing to permit long-term water level observation. It is anticipated that soft clay soils may be encountered at this site. If so, an additional 4-inch diameter cased hole may be required to extract undisturbed tube samples and/or perform in situ vane shear tests. Before the drill crew demobilizes, the driller should telephone the results of the first three SPT borings to the project engineer, Mr. Richard Cheney at 202-426-0355. At that time, a decision on the details of the UDH will be issued.

A wet area of potentially unstable soil (soft and/or organic soils) exists in the area of the proposed east approach embankment. Please define the depth of this deposit beneath the limits of the east approach embankment back to Baseline station 93 + 50 with hand auger exploration.

The present schedule for structure design requires that all samples and subsurface logs be received in the main office by July 1, 1992.

Attachment: Proposed site exploration plan

Apple Freeway Design Example – Site Exploration
Exhibit C – Typical Boring Request

REGION 3
COUNTY Orange
PROJECT Interstate 0
DATE START 5/2/92
DATE FINISH 5/3/92
CASING O.D. 2-1/2"I.D.
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

**SUBSURFACE EXPLORATION LOG**

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-1
LINE Baseline
STA. 90+77
OFFSET 50' Rt.
SURF. ELEV. 1001.1

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/2/92 | 5/3/92 | |
| DEPTH TO WATER | 15' | 15' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | Sample Recovery/in | DESCRIPTION OF SOIL AND ROCK | MOIST CONT % |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 4 | J1 | 1 | 3 | 5 | | | | 10 |
| | 16 | | | | | | | | |
| | 25 | | | | | | | | |
| | 16 | | | | | | | | |
| | 20 | | | | | | | | |
| | 30 | J2 | 7 | 7 | 8 | | | GR. FINE TO COARSE SAND | 7 |
| | 21 | | | | | | | | |
| | 35 | | | | | | | MOIST - NON PLASTIC | |
| | 38 | | | | | | | | |
| 10 | 51 | | | | | | | | |
| | 40 | J3 | 10 | 21 | 20 | | | | 6 |
| | 35 | | | | | | | | |
| | 52 | | | | | | | | |
| | 58 | | | | | | | | |
| | 61 | | | | | | | | |
| | 50 | J4 | 10 | 18 | 21 | | | | 6 |
| | 42 | | | | | | | | |
| | 65 | | | | | | | | |
| | 72 | | | | | | | | |
| 20 | 76 | | | | | | | 20' | |
| | 60 | J5 | 3 | 6 | 6 | | | | 31 |
| | 51 | | | | | | | | |
| | 72 | | | | | | | | |
| | 75 | | | | | | | GR. SILTY CLAY | |
| | 81 | | | | | | | | |
| | 80 | J6 | 3 | 6 | 7 | | | MOIST - PLASTIC | 32 |
| | 71 | | | | | | | | |
| | 90 | | | | | | | | |
| | 83 | | | | | | | | |
| 30 | 84 | | | | | | | | |
| | 80 | J7 | 2 | 4 | 4 | | | | 36 |
| | 77 | | | | | | | | |
| | 83 | | | | | | | | |
| | 84 | | | | | | | | |
| | 86 | | | | | | | | |

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CONTRACTOR ________ SM ________

DRILL RIG OPERATOR Klinedinst
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Chenev
SHEET 1 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-1

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs

REGION 3
COUNTY Orange
PROJECT Interstate 0
DATE START 5/2/92
DATE FINISH 5/3/92
CASING O.D. 2-1/2" I.D.
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-1
LINE Baseline
STA. 90+77
OFFSET 50' Rt.
SURF. ELEV. 1001.1

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/2/92 | 5/3/92 | |
| DEPTH TO WATER | 15' | 15' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST CONT % |
|---|---|---|---|---|---|---|---|---|
| 35 | 80 | J8 | 2 | 3 | 3 | | | 38 |
| | 76 | | | | | | GR. SILTY CLAY | |
| | 85 | | | | | | MOIST - PLASTIC | |
| | 87 | | | | | | | |
| | 89 | | | | | | 40.5' | |
| 40 | 81 | | 3 | 3 | 50 | | | |
| | 120 | | | | | | | |
| | 140 | | | | | | GR. SANDY GRAVEL | |
| | 156 | | | | | | | |
| | 180 | | | | | | MOIST - NON PLASTIC | |
| | 160 | | 35 | 50 | | | | |
| | 195 | | | | | | | |
| | 200 | | | | | | | |
| | 210 | | | | | | | |
| | 230 | | | | | | TOP OF ROCK 50'Z | |
| 50 | | | | | | | HARD. UNWEATHERED BASALT | |
| | | | | | | | Run 1 = 60" | |
| | | | | | | | RECOVERY = 48" RQD = 70% | |
| | | | | | | | 9 PIECES | |
| | | | | | | | 55' | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| | | | | | | | RUN 2 = 60" RQD = 80% | |
| | | | | | | | RECOVERY = 54" 8 PIECES | |
| 60 | | | | | | | 60' | |
| | | | | | | | END OF BORING 60' | |

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CONTRACTOR ______ SM ______

DRILL RIG OPERATOR Klinedinst
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 2 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-1

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

REGION 3
COUNTY ORANGE
PROJECT INTERSTATE 0
DATE START 5/4/92
DATE FINISH 5/6/92
CASING O.D. 2-1/2" I.D.
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

# SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-2
LINE Baseline
STA. 92+00
OFFSET 50' Lt.
SURF. ELEV. 996.2

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/4/92 | 5/6/92 | |
| DEPTH TO WATER | 10' | 10' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST. CONT. % |
|---|---|---|---|---|---|---|---|---|
| 0 | 6 | J1 | 1 | 2 | 2 | | | 12 |
| | 19 | | | | | | | |
| | 27 | J2 | 1 | 3 | 3 | | GR - SILTY SAND | 8 |
| | 35 | | | | | | | |
| | 21 | J3 | 2 | 5 | 6 | | MOIST - NON PLASTIC | 7 |
| | 30 | J4 | 7 | 9 | 12 | | | 10 |
| | 22 | | | | | | | |
| | 25 | J5 | 8 | 7 | 15 | | | 6 |
| | 24 | | | | | | | |
| 10 | 28 | J6 | 14 | 20 | 20 | | | 7 |
| | 27 | J7 | 15 | 18 | 19 | | | |
| | 36 | | | | | | | 10 |
| | 34 | J8 | 13 | 16 | 17 | | | 7 |
| | 37 | | | | | | 14.5' | |
| | 39 | J9 | 15 | 10 | 3 | | | 34 |
| | 31 | | | | | | | |
| | 40 | | | | | | | |
| | 46 | | | | | | | |
| | 46 | | | | | | | |
| 20 | 45 | | | | | | | |
| | 41 | J10 | 2 | 4 | 4 | | | 31 |
| | 42 | | | | | | GR. SILTY CLAY | |
| | 56 | | | | | | | |
| | 52 | | | | | | MOIST - PLASTIC | |
| | 58 | | | | | | | |
| | 50 | J11 | 2 | 3 | 3 | | | 36 |
| | 56 | | | | | | | |
| | 52 | | | | | | | |
| | 49 | | | | | | | |
| 30 | 58 | | | | | | | |
| | 52 | J12 | 1 | 2 | 3 | | | 37 |
| | 56 | | | | | | | |
| | 61 | | | | | | | |
| | 63 | | | | | | | |
| | 65 | | | | | | | |

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CONTRACTOR ______ SM ______

DRILL RIG OPERATOR Klinedinst
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 1 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-2

Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

REGION 3
COUNTY ORANGE
PROJECT INTERSTATE 0
DATE START 5/4/92
DATE FINISH 5/6/92
CASING O.D. 2-1/2" I.D.
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-2
LINE Baseline
STA. 92+00
OFFSET 50' Lt.
SURF. ELEV. 996.2

| TIME | 4 pm | 8 am | |
|---|---|---|---|
| DATE | 5/4/92 | 5/6/92 | |
| DEPTH TO WATER | 10' | 10' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST. CONT. % |
|---|---|---|---|---|---|---|---|---|
| 35 | 62 | J13 | 1 | 2 | 4 | | GR SILTY CLAY | 34 |
| | 63 | | | | | | MOIST - PLASTIC | |
| | 72 | | | | | | | |
| | 71 | | | | | | 39' | |
| 40 | 89 | | | | | | | |
| | 75 | | 30 | 20 | 35 | | | |
| | 81 | | | | | | GR SANDY GRAVEL | |
| | 401 | | | | | | | |
| | 120 | | | | | | MOIST - NON PLASTIC | |
| | 140 | | | | | | (Cored Boulder 42.5' to 45', recovered 12"-7 pieces; | |
| | 110 | | | 50 | 60 | | used boulder buster) | |
| | 136 | | | | | | | |
| | 166 | | | | | | | |
| | 191 | | | | | | | |
| 50 | 482 | | | | | | Top of Rock 50' | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| | | | | | | | Run 1 - 60" | |
| | | | | | | | Recovery - 55" - 13 pieces RQD = 80% | |
| | | | | | | | 55' | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| | | | | | | | Run 2 - 60" | |
| | | | | | | | Recovery - 52" 7 pieces RQD - 75% | |
| | | | | | | | 60' | |
| 60 | | | | | | | end of boring 60' | |

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SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 2 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-2

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

REGION 3
COUNTY ORANGE
PROJECT INTERSTATE 0
DATE START 5/8/92
DATE FINISH 5/9/92
CASING O.D. 2-1/2" I.D.
SAMPLER O.D. 2" I.D. 1-1/2"
RIG TYPE Acker B-40
CORE BARREL Double Tube

SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-3
LINE Baseline
STA. 93+27
OFFSET 50' Rt.
SURF. ELEV. 990.0

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/8/92 | 5/9/92 | |
| DEPTH TO WATER | 6' | 6' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST. CONT. % |
|---|---|---|---|---|---|---|---|---|
| 0 | 0 | J1 | 1 | 0 | 1 | | BLACK MUCK WET - PLASTIC | 115 |
| | 2 | | | | | | 2' | |
| | 11 | J2 | 3 | 5 | 7 | | | 20 |
| | 25 | | | | | | | |
| | 31 | | | | | | GR SAND W/ ROOTS AND FIBERS | |
| | 40 | | | | | | | |
| | 41 | J3 | 8 | 8 | 9 | | MOIST - NON PLASTIC | 8 |
| | 56 | | | | | | | |
| | 71 | | | | | | | |
| 10 | 83 | | | | | | 10' | |
| | 70 | J4 | 6 | 5 | 5 | | | 29 |
| | 91 | | | | | | | |
| | 93 | | | | | | | |
| | 82 | | | | | | GR-BR CLAYEY SILT | |
| | 93 | | | | | | | |
| | 81 | J5 | 2 | 3 | 6 | | MOIST PLASTIC | 31 |
| | 80 | | | | | | | |
| | 87 | | | | | | | |
| | 85 | | | | | | | |
| 20 | 90 | | | | | | 20' | |
| | 82 | J6 | 4 | 3 | 3 | | | 34 |
| | 86 | | | | | | | |
| | 87 | | | | | | GR SILTY CLAY | |
| | 85 | | | | | | | |
| | 90 | | | | | | MOIST - PLASTIC | |
| | 73 | J7 | 2 | 2 | 3 | | | 39 |
| | 72 | | | | | | | |
| | 83 | | | | | | | |
| | 71 | | | | | | | |
| 30 | 61 | | | | | | | |
| | 81 | J8 | 2 | 2 | 2 | | | 40 |
| | 83 | | | | | | | |
| | 72 | | | | | | | |
| | 76 | | | | | | | |
| | 83 | | | | | | | |

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SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 1 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-3

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

REGION 3
COUNTY ORANGE
PROJECT INTERSTATE 0
DATE START 5/8/92
DATE FINISH 5/9/92
CASING O.D. 2-1/2" I.D.
SAMPLER O.D. 2" I.D. 1-1/2"
RIG TYPE Acker B-40
CORE BARREL Double Tube

SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-3
LINE Baseline
STA. 93+27
OFFSET 50'Rt.
SURF. ELEV. 990.0

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/8/92 | 5/9/92 | |
| DEPTH TO WATER | 6' | 6' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST. CONT. % |
|---|---|---|---|---|---|---|---|---|
| 35 | 71 | J9 | 2 | 3 | 2 | | | 36 |
| | 79 | | | | | | GR SILTY CLAY | |
| | 86 | | | | | | | |
| | 83 | | | | | | MOIST — PLASTIC | |
| | 85 | | | | | | | |
| 40 | 82 | J10 | 3 | 4 | 3 | | | 35 |
| | 81 | | | | | | | |
| | 93 | | | | | | | |
| | 91 | | | | | | | |
| | 96 | | | | | | 45' | |
| | 121 | J11 | 20 | 21 | 35 | | | 10 |
| | 450 | | | | | | GR SILTY GRAVEL | |
| | 391 | | | | | | | |
| | 220 | | | | | | MOIST - NON PLASTIC | |
| 50 | 230 | | | | | | | |
| | 200 | J12 | 15 | 36 | 40 | | 52' to 53' CORED BOULDER | 5 |
| | 370 | | | | | | RECOVERY 3" | |
| | 400 | | | | | | MANY FRAGMENTS | |
| | 410 | | | | | | | |
| | 380 | | | | | | | |
| | | J13 | 40 | 60 | 80 | | | 7 |
| 60 | | | | | | | TOP OF ROCK 60.5' | |
| | | J14 | 100 | REFUSAL @ 60.5' | | | HARD UNWEATHERED BASALT | |
| | | | | | | | RUN 1 60.5' to 65.5' - 60" - RECOVERY 50", 12 PIECES RQD | 70% |
| | | | | | | | RUN 2 65.5' to 70.5' - 60" - RECOVERY 60", 6 pieces RQD 95% | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| 70 | | | | | | | | |
| | | | | | | | End of Boring 70.5' | |

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SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 2 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-3

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

REGION 3
COUNTY ORANGE
PROJECT INTERSTATE 0
DATE START 5/10/92
DATE FINISH 5/12/92
CASING O.D. 4" I.D.
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

## SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-4
LINE Baseline
STA. 93+27
OFFSET 50' Lt.
SURF. ELEV. 991.0

| TIME | 4 pm | 8 am | |
|---|---|---|---|
| DATE | 5/10/92 | 5/12/92 | |
| DEPTH TO WATER | 6' | 6' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST CONT % |
|---|---|---|---|---|---|---|---|---|
| 0 | 0 | J1 | 1 | 1 | 1 | | BLACK ORGANIC SILT | 120 |
| | 2 | | | | | | WET - PLASTIC | |
| | 11 | | | | | | 3' | |
| | 25 | | | | | | | |
| | 31 | | | | | | | |
| | 40 | J2 | 4 | 8 | 9 | | GR. SAND | 12 |
| | 41 | | | | | | | |
| | 56 | | | | | | MOIST - NON PLASTIC | |
| | 71 | | | | | | | |
| 10 | 83 | | | | | | 10' | |
| | 70 | T3 | | | | | (10' - 12' PUSHED TUBE 20") | 33 |
| | 91 | | | | | | | |
| | 93 | | | | | | | |
| | 82 | | VANE | | | | (13' VANE SHEAR TEST | |
| | 93 | | | | | | | |
| | 81 | T4 | | | | | (15' - 17' PUSHED TUBE 20") | 35 |
| | 80 | | | | | | | |
| | 87 | | | | | | GR. CLAYEY SILT | |
| | 85 | | VANE | | | | (18' VANE SHEAR TEST. | |
| 20 | 90 | | | | | | MOIST - PLASTIC | |
| | 82 | T5 | | | | | (20' - 22' PUSHED TUBE 20") | 31 |
| | 86 | | | | | | | |
| | 87 | | | | | | | |
| | 85 | | VANE | | | | (23' VANE SHEAR TEST | |
| | 75 | | | | | | | |
| | 73 | T6 | | | | | (25'-27' PUSHED TUBE 20") | 36 |
| | 72 | | | | | | | |
| | 83 | | | | | | | |
| | 71 | | VANE | | | | (28' VANE SHEAR TEST) | |
| 30 | 61 | | | | | | | |
| | 81 | T7 | | | | | (30' - 32' PUSHED TUBE 20") | 38 |
| | 83 | | | | | | | |
| | 72 | | | | | | | |
| | 76 | | | | | | | |
| | 83 | | | | | | | |

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SHEET 1 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-4

Apple Freeway Design Example – Site Exploration
Exhibit D – Boring Logs (Cont'd)

| | | |
|---|---|---|
| REGION 3 | **SUBSURFACE EXPLORATION LOG** | HOLE BAF-4 |
| COUNTY ORANGE | | LINE Baseline |
| PROJECT INTERSTATE 0 | | STA. 93+27 |
| DATE START 5/10/92 | HAMMER FALL-CASING 18" | OFFSET 50' Lt. |
| DATE FINISH 5/12/92 | HAMMER FALL-SAMPLER 30" | SURF. ELEV. 991.0 |
| CASING O.D. 4" I.D. | WEIGHT OF HAMMER-CASING 300 LBS. | |
| SAMPLER O.D. 2" I.D. 1-3/8" | WEIGHT OF HAMMER-SAMPLER 140 LBS. | |
| RIG TYPE Acker B-40 | | |
| CORE BARREL Double Tube | | |

| | | | |
|---|---|---|---|
| TIME | 4 pm | 8 am | |
| DATE | 5/10/92 | 5/12/92 | |
| DEPTH TO WATER | 5' | 5' | |

| DEPTH BELOW SURFACE | BLOWS ON CASING | SAMPLE NO. | BLOWS ON SAMPLER 0/.5 | .5/1.0 | 1.0/1.5 | 1.5/2.0 | DESCRIPTION OF SOIL AND ROCK | MOIST CONT % |
|---|---|---|---|---|---|---|---|---|
| 36 | 71 | J8 | 2 | 2 | 4 | | GR CLAYEY SILT | 38 |
| | 79 | | | | | | | |
| | 86 | VANE | | | | | (37' VANE SHEAR TEST) MOIST - PLASTIC | |
| | 83 | | | | | | | |
| 40 | 85 | | | | | | | |
| | 82 | | | | | | (40' - 42' PUSHED TUBE 18") | 37 |
| | 81 | T9 | | | | | | |
| | 93 | | | | | | | |
| | 91 | | | | | | 45' | |
| | 96 | | | | | | | |
| | 121 | J10 | 7 | 8 | 15 | | | |
| | 450 | | | | | | | |
| | 391 | | | | | | GR SANDY GRAVEL | |
| | 220 | | | | | | | |
| 50 | 230 | | | | | | MOIST - NON PLASTIC | |
| | 200 | J11 | 40 | 100 | | | | |
| | 370 | | | | | | | |
| | 400 | | | | | | | |
| | 410 | | | | | | TOP OF ROCK — 55' | |
| | 380 | | | | | | | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| | | | | | | | Run 1 - 55' - 60' (60") RECOVERY 58", 21 PIECES RQD 90% | |
| | | | | | | | 60' | |
| 60 | | | | | | | | |
| | | | | | | | Run 2 - 60' - 65' (60") RECOVERY - 60", 5 PIECES RQD 95% | |
| | | | | | | | HARD UNWEATHERED BASALT | |
| | | | | | | | end of boring 65' | |
| 70 | | | | | | | | |

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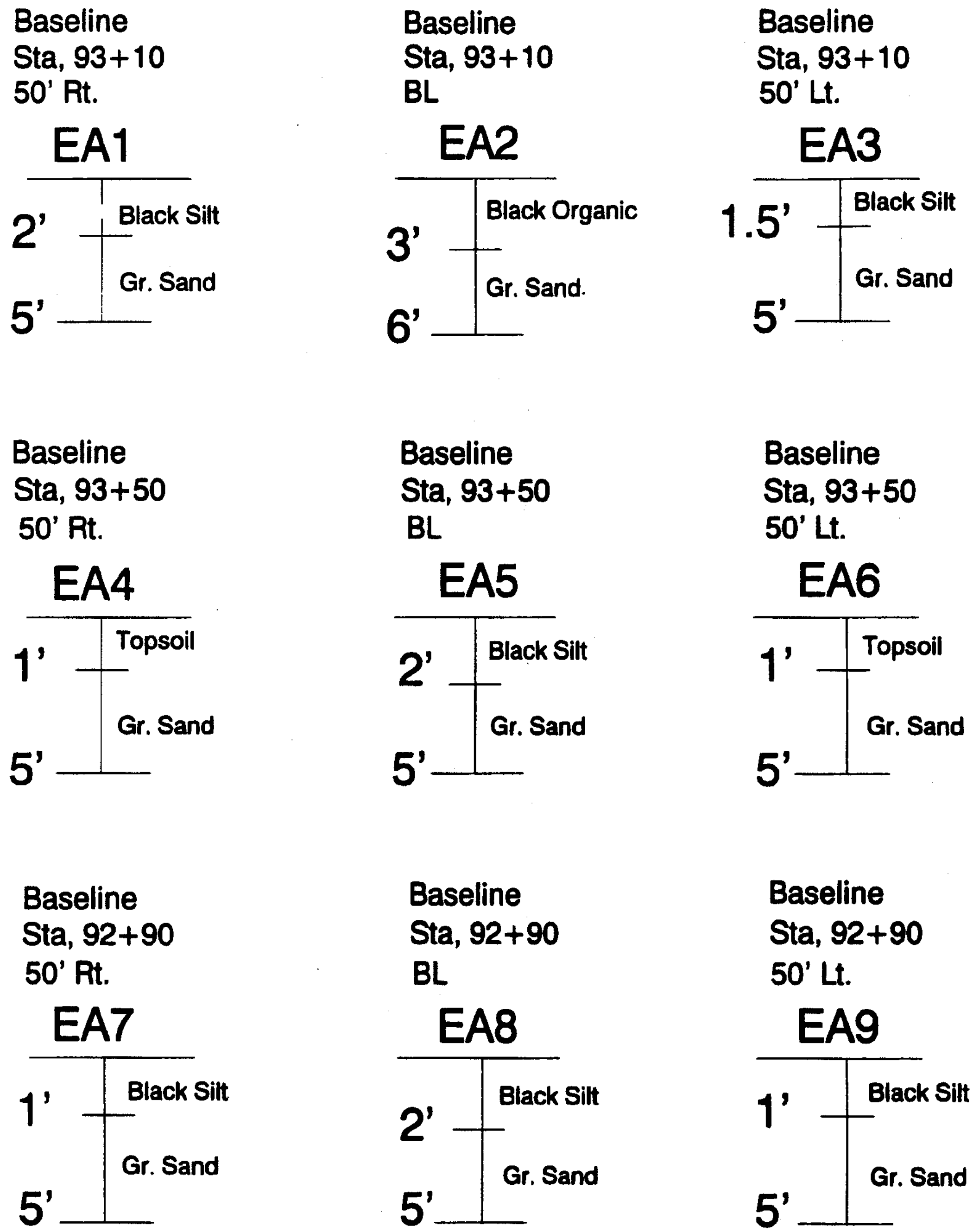
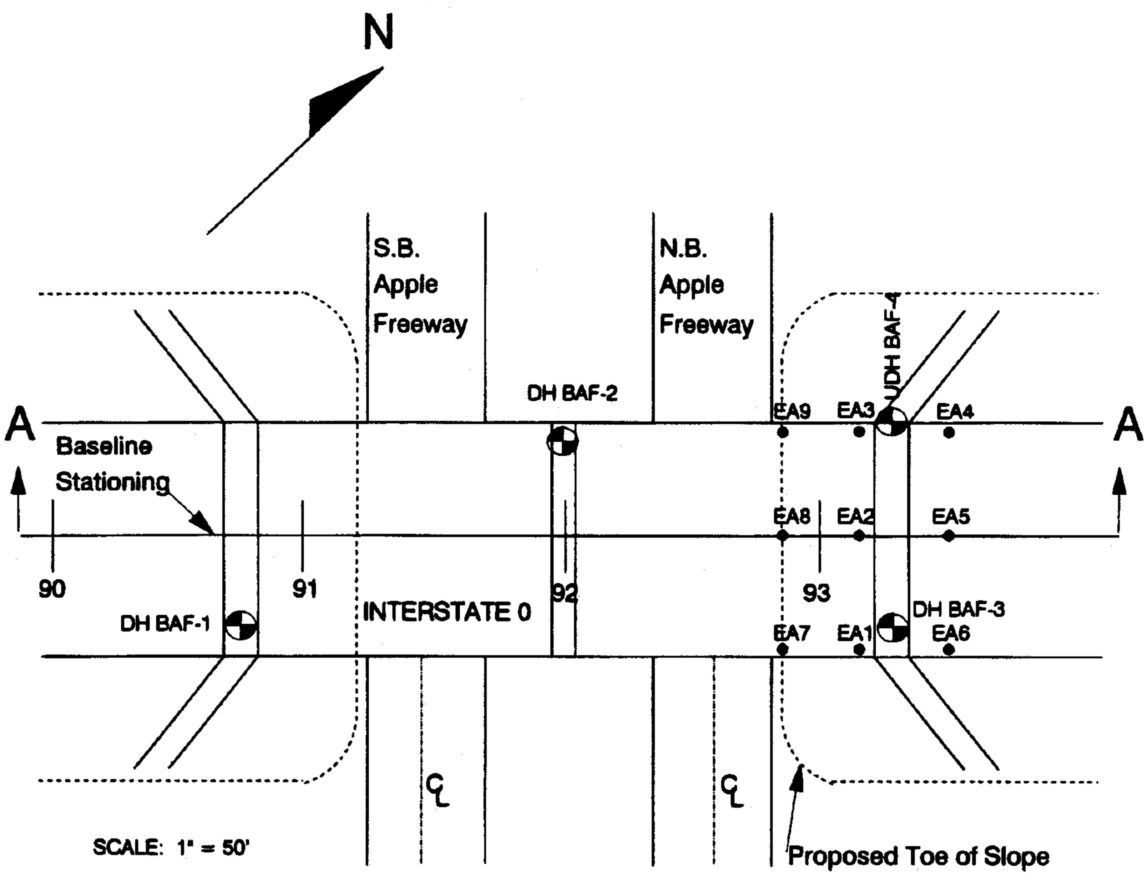
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 2 OF 2
STRUCTURE NAME/NO. Apple Freeway #2
HOLE BAF-4

Apple Freeway Design Example – Site Exploration
Exhibit D - Boring Logs (Cont'd)




Apple Freeway Design Example – Site Exploration
Exhibit E – East Abutment Area Hand Auger Hole Logs




Apple Freeway Design Example – Site Exploration
Exhibit F – Final Exploration Locations

**Summary of the Site Exploration Phase for Apple Freeway Design Problem**

- Terrain Reconnaissance

  Delta landform - possible clay deposit buried

- Site Inspection

  Unsuitable soils near east approach embankment

- Subsurface Borings

  Hand auger holes define limits and depth of unsuitable organic deposit.

  SPT drill holes show sand over clay over gravel and rock.

  Undisturbed samples and vane shear tests taken in clay.

# CHAPTER 3.0
# BASIC SOIL PROPERTIES FOR FOUNDATION DESIGN

The foundation engineer is usually concerned with the construction of some type of engineering structure on or in the earth. For engineering purposes, we shall consider the earth to be made up of rock and soil. Rock is that naturally occurring material composed of mineral particles so firmly bonded together that relatively great effort is required to separate the particles (i.e., blasting or heavy crushing forces). Soil will be defined as naturally occurring mineral particles which are fairly readily separated into relatively small pieces, and in which the mass may contain air, water, or organic materials (derived from decay of vegetation, etc.). The mineral particles of the soil mass are formed from decomposition of the rock by weathering (by air, ice, wind, and water) and chemical processes. Classification of soils by particle size according to various standards is shown in Figure 3 – 1.

| MAIN SOIL GROUPS | SOIL TYPES |
|---|---|
| Granular Soils | Sands and Gravels |
| Fine-Grained Soils | Silts and Clays |
| Organic Soils | Peat, Organic Clays, and Organic Silts |

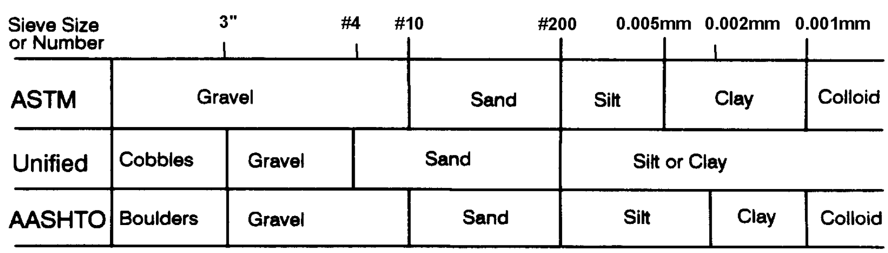



Figure 3 – 1: Particle Size Limit by Different Classifications Systems

## 3.1 ENGINEERING PROPERTIES OF SOILS

The major engineering properties of the main soil groups as related to foundation design are summarized as follows:

### 3.1.1 Engineering Properties of Granular Soils

- Excellent foundation material for supporting structures and roads.
- The best embankment material.
- The best backfill material for retaining walls.
- Might settle under vibratory loads or blasts.
- Dewatering can be difficult due to high permeability.

- If free draining not frost susceptible.

### 3.1.2 Engineering Properties of Cohesive Soils

- Very often possess low shear strength.
- Plastic and compressible.
- Loses part of shear strength upon wetting.
- Loses part of shear strength upon disturbance.
- Shrinks upon drying and expands upon wetting.
- Very poor material for backfill.
- Poor material for embankments.
- Practically impervious.
- Clay slopes are prone to landslides.

### 3.1.3 Engineering Properties of Silt

- Relatively low shear strength
- High capillarity and frost susceptibility
- Relatively low permeability
- Difficult to compact

**Engineering Properties of Silt as Compared to Clay**

- Better load sustaining qualities
- Less compressible
- More permeable
- Exhibits less volume change

### 3.1.4 Engineering Properties of Organic Soils

Any soil containing a sufficient amount of organic matter to influence its engineering properties is called an organic soil. The term organic designates those soils containing an appreciable amount of decayed animal and/or vegetative matter in various states of decomposition.

The organic matter is objectionable for three main reasons:

1. Reduces load-carrying capacity of soil.

2. Increases compressibility considerably.

3. Frequently contains toxic gasses that are released during the excavation process.

All organic soils, whether peat, organic clays, organic silts, or even organic sands, should be viewed with suspicion as foundation and construction materials.

## 3.2 GRANULAR MATERIAL PROPERTIES

Grain size distribution is the single most important element in the design of granular material items. Grain size distribution is determined by sieving a soil sample of known weight through U.S. Standard mesh opening sizes. The percentages of total sample are recorded and plotted on a semi-log sheet (Figure 3 – 2). The resulting curves represent the grain size distribution in the soil sample.

Much can be learned about a sample's engineering properties from the shape and location of the curve. For instance, the well-graded curve represents a soil sample with a wide range of particle sizes that are evenly distributed. Densification of a well-graded sample causes the small particles to move into the voids between larger particles. As the voids in the sample are reduced, the density and strength of the sample increase. Specifications for select structural fill should contain required ranges of different particle sizes so that a dense, non-compressible backfill can be achieved with minimal compactive effort. For example, the well-graded material shown in Figure 3-2 could be specified by providing the following gradation limits:

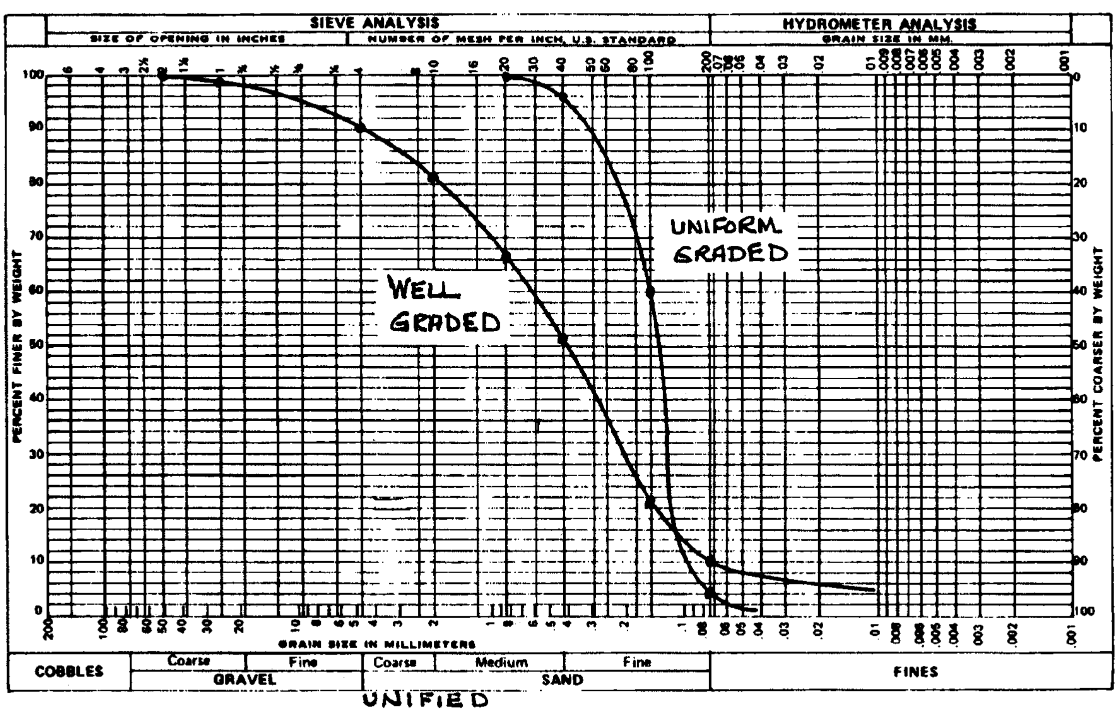



Figure 3 – 2: Grain Size Distribution

**TABLE 3 – 1**
**TYPICAL GRADATION LIMITS OF WELL-GRADED GRANULAR MATERIAL**

| Sieve Size | Percent Passing by Weight |
|---|---|
| 2″ | 100 |
| #10 | 75-90 |
| #40 | 40-60 |
| #200 | less than 15 |

A uniform graded material is composed of a narrow range of particle sizes. When compaction is attempted, inadequate distribution of particle sizes prevents reduction of the volume of voids in the soil. Such uniform materials should be avoided as select fill material. However, uniform graded materials do have an important use as drainage material. The relatively large void spaces act as conduits to carry water. Obviously, the larger the average particle size, the larger the void space. The early "French" drain was an example of the use of a coarse uniform graded soil. Typical specifications for drainage materials would show a narrow band of particle sizes:

**TABLE 3 – 2**
**TYPICAL GRADATION LIMITS OF DRAINAGE MATERIALS**

| Sieve Size | Percent Passing by Weight |
|---|---|
| 2″ | 100 |
| 1 ½ ″ | 90-100 |
| 1″ | 0-15 |

The durability of aggregates is also an important item in specifications. Non-durable materials tend to breakdown which causes a change in the grain size distribution. Smaller grains will tend to reduce the size of the mass and result in surface settlement or in the case of drainage material, clogging of the drainage paths. Infiltration of fines into drainage aggregate also causes clogging. Typically modern drainage aggregate systems are wrapped in a suitable geotextile to prevent contamination of the aggregate.

## 3.3 FINE-GRAINED MATERIAL PROPERTIES

Another important concept is that of plasticity of soils. During a visual examination of soil samples containing fine-grained materials, a judgment is made that the soil is plastic, or non-plastic but no relative value is assigned. Arbitrary indices have been chosen to define the plasticity of cohesive (clay) soils (Table 3 - 3). These are liquid limit (LL), plastic limit (PL), and plasticity index (PI). These limits quantitatively describe the effect of varying water content on the consistency of fine-grained soils. With increasing water content, fine-grained soils pass consecutively from the solid to semi-solid to plastic to liquid states. These limits and the applicable standard AASHTO test numbers are shown in Figure 3 - 3 and Table 3 – 3.

**TABLE 3 – 3**
**SOIL PLASTICITY CHARACTERISTICS**

| Plasticity Characteristics | Symbol | Units′ | How Obtained | Application |
|---|---|---|---|---|
| Liquid limit | LL | D | Directly from test AASHTO T89 | Classification & properties correlation. |
| Plastic limit | PL | D | Directly from test AASHTO T89 | Classification. |
| Plastic index | PI | D | LL-PL | Classification & properties correlation |
| Shrinkage limit | SL | D | Directly from test AASHTO T89 | Classification computation of swell |
| Shrinking index | SI | D | PL-SL | |
| Activity | Ac | D | $\frac{\text{PI}}{\%\ \text{"Clay Size"}}$ | Identification of clay mineral |
| Liquidity index | LI | D | $\frac{\text{W} - \text{PL}}{\text{PI}}$ | Estimating degree of preconsolidation |

Units′: D = Dimensionless

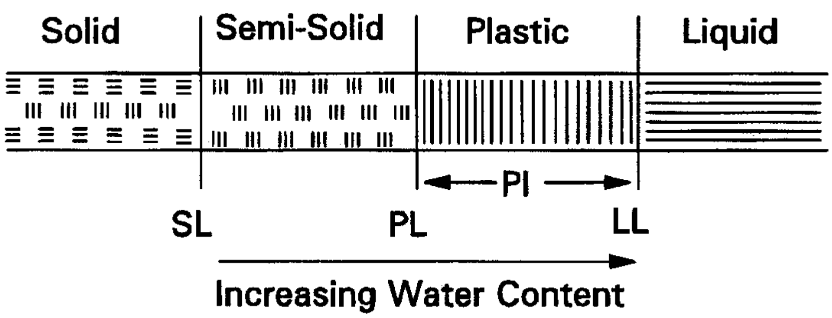



Figure 3 - 3: Relationship between Soil State and Atterberg Limit

The plasticity index (PI) represents the range of water content in which the soil remains plastic. In general, the plasticity index represents the relative amount of clay particles in the soil. The higher the PI, the greater the amount of clay particles present, and the more plastic the soil. A more plastic soil will:

1. Be more compressible.

2. Have higher shrink-swell potential.

3. Be less permeable.

Atterberg limits are a <u>cheap</u> method of obtaining a lot of useful data.

## 3.4 SOIL IDENTIFICATION, DESCRIPTION AND CLASSIFICATION

Three terms, which are used in the site exploration process, are: IDENTIFY, DESCRIBE, and CLASSIFY. Identification is the process of determining which components exist in a particular soil sample, i.e., gravel, sand, silt, clay, etc. Description is the process of estimating the relative percentage of each component and preparing a word picture of the sample (ASTM D2488). Identification and description are accomplished primarily by both a visual examination and the feel of the sample.

Classification is the process of grouping soils with similar engineering properties into categories. For example, the Unified Soil Classification System (ASTM D2487), which is the most commonly used system in geotechnical work, is based on grain size, gradation, and plasticity. The AASHTO system (M145), which is also of interest to highway engineers, groups soils into categories having similar load carrying capacity and service characteristics for pavement subgrade design.

The important distinction between classification and both identification and description is that standard AASHTO or ASTM laboratory tests must be performed to determine a soil's classification. Highway agencies typically do not need to perform the laboratory tests necessary to classify every soil sample. Instead soil technicians are trained to accurately identify and describe soil samples to an accuracy which is acceptable for highway engineering work.

During progression of a boring, the field drilling personnel should only roughly identify and describe the soils encountered. Unfortunately the drillers are usually delegated the task of exactly identifying and describing the soil samples. This is unfair, as drillers must be concerned with many other tasks involving mechanical operation of the rig and preparation of pertinent data for the subsurface log. ln addition, the visual identification test should not be done outdoors in an atmosphere subjected to the elements, as this single operation will provide the basis for later testing and soil profile development. Instead, the soil samples should be sent to a laboratory and visually identified by a technician experienced in soils work. This is of great importance where no laboratory testing is to be performed and design values are estimated on the visual description and SPT results.

The identification system used should permit the engineer to easily relate the soil description to its appearance and behavior characteristics. Density of granular soils or consistency of cohesive soils may be estimated from SPT N-values as previously described in Table 2 – 3. Classification tests, except for moisture content, may be performed on typical samples to verify identification. If possible, the moisture content of every sample should be determined. A typical soil description procedure, known as the Modified Unified Description (MUD), is shown in Appendix A. The Unified Classification System is shown in Appendix B.

## 3.5 ROCK CLASSIFICATION*

Rock is classified with respect to its geological origin as follows:

- Igneous rocks – such as granite, diorite and basalt, are those formed by the solidification of molten material, either by intrusion at depth in the earth's crust or by extrusion at the earth's surface.

- Sedimentary rocks – such as sandstone, limestone and shale, are those rocks formed by deposition, usually under water, of products derived by the disaggregation of pre-existing rocks.

* Based on: Canadian Foundation Engineering Manual (March, 1978).

- Metamorphic rocks – such as quartzite, schist and gneiss, may be either igneous or sedimentary rocks which have been altered physically and sometimes chemically by the application of intense heat and pressure at some time in their geological history

### 3.5.1 Structural Features of Rock Masses

Geological structures generally have a significant influence on the rock mass properties. Some of the important features are described as follows:

- Rock mass – means an aggregate of blocks of solid rock material containing structural features, which constitute mechanical discontinuities. Rock mass refers to any in situ rock with all inherent geomechanical discontinuities.

- Rock material – or intact rock means the consolidated aggregate of mineral particles forming solid material between structural discontinuities. Properties attributed to it refer to rock material free of geomechanical discontinuities.
- Geomechanical or structural discontinuities – means all geological features which separate solid blocks of the rock mass, such as joints, faults, bedding planes, cleavage planes, shear zones, and solution cavities. These features constitute planes of weakness which reduce the strength of the rock mass appreciably.

- Major discontinuities or major structures – means those geological features constituting structural discontinuities which are sufficiently well developed and continuous that shear failure along them would involve little or no shearing of intact rock material.

### 3.5.2 Engineering Properties of Rock Masses

The quality of a rock mass for foundation purposes depends mainly upon the strength of rock material and on the spacing, the nature (width, roughness, waviness, weathering, etc.) and the orientation of discontinuities. Classification of rock according to some of those properties is given in Table 3-3 and 3-4.

**TABLE 3 - 4**
**CLASSIFICATION OF ROCK WITH RESPECT TO STRENGTH**

| Classification of Rock with Respect to Strength | Unconfined Compressive Strength - PSI |
|---|---|
| Very high strength | greater than 32,000 |
| High strength | 8,000 to 32,000 |
| Medium strength | 2,000 to 8,000 |
| Low strength | 500 to 2,000 |
| Very low strength | 125 to 500 |

Note: Rocks with compressive strengths lower than 125-lb/sq. in. should be treated as soils.

**TABLE 3 - 5**
**CLASSIFICATION OF ROCK MASS WITH RESPECT TO THE SPACING OF DISCONTINUITIES**

| Classification of Rock Mass with Respect to the Spacing of Discontinuities | Average Spacing |
|---|---|
| Very wide | greater than 10 ft |
| Wide | 3 ft. to 10 ft. |
| Moderately close | 1 ft. to 3 ft. |
| Close | 2 in. to 1 ft. |
| Very close | smaller than 2 in. |

### 3.5.3 Nature and Orientation of Rock Discontinuities

For foundation purposes, the nature of rock discontinuities may be expressed in terms of their width, the degree of weathering of rock contact faces, and the character of infilling materials.

In addition to the strength of rock material, and the spacing and nature of discontinuities, the quality of a rock mass for foundation purposes is affected by the orientation of discontinuities with respect to the applied load. A rock mass is said to contain adversely oriented discontinuities, if under the action of the resultant foundation load the minimum resistance to sliding occurs when the sliding surface is considered to be along these discontinuities.

### 3.5.4 Rock Quality Designation (RQD)

This is a general method by which the quality of the rock at a site, is obtained based on the relative amount of fracturing and alteration.

The Rock Quality Designation (RQD) is based on a modified core recovery procedure which, in turn, is based indirectly on the number of fractures (except those due directly to drilling operations) and the amount of softening or alteration in the rock mass as observed in the rock cores from a drill hole. Instead of counting the fractures, an indirect measure is obtained by summing the total length of core recovered by counting only those pieces of hard and sound core which are 4 inches or greater in length. The ratio of this modified core recovery length to the total core run length is known as the RQD.

An example is given below from a core run of 60 inches. For this particular case the total core recovery is 50 inches yielding a core recovery of 83 percent. On the modified basis, only 38 inches are counted and the RQD is 63 percent.

| CORE RECOVERY, in | MODIFIED CORE RECOVERY, in |
|---|---|
| 10 | 10 |
| 2 | |
| 2 | |
| 3 | |
| 4 | 4 |
| 5 | 5 |
| 3 | |
| 4 | 4 |
| 6 | 6 |
| 4 | 4 |
| 2 | |
| 5 | 5 |
| Total = 50 | Total = 38 |

Therefore, Percentage Core Recovery = 50/60 = 83%; RQD= 38/60 = 63%

A general description of the rock quality can be made from the RQD Value (Table 3-5).

**TABLE 3 - 6**
**RQD DESCRIPTION**

| RQD (ROCK QUALITY DESIGNATION) | DESCRIPTION OF ROCK QUALITY |
|---|---|
| 0 – 25 | Very poor |
| 26 – 50 | Poor |
| 51 – 75 | Fair |
| 76 – 90 | Good |
| 91 – 100 | Excellent |

## 3.6 SOIL PROFILE DEVELOPMENT

The mark of successfully accomplishing a subsurface investigation is the ability to draw a soil profile of the project site complete with soil types and necessary design properties. The soil profile is a visual display of subsurface conditions as interpreted from all foregoing explorations and testing. Uncertainties in its development usually indicate additional explorations or testing are required.

In the optimum situation the soil profile is developed in stages. First, a rough profile is established from the drillers' logs by the soils engineer or geologist. The object is to discover any obvious gaps or question marks while the drill crew is still at the site so that additional work can be performed immediately. Once a crew has left the site, a delay of months may occur before their schedule permits reoccupying the site (not to mention the additional cost to the highway agency, and aggravation to the drill crew to reoccupy a remote site). The drilling inspector or crew chief should be required to call the soils engineer when progression of the last scheduled boring has begun, to request further instructions for supplemental borings.

When all borings are completed and laboratory visuals and moisture content data received, the initial soil profile should be revised. Definite soil layer boundaries and accurate soil descriptions should be established for soil deposits. Too often the engineer will over-complicate a simple profile by noting minute variations between adjacent soil samples. This can be avoided by:

1. Reviewing the geologic site history, i.e., if the soil map denotes a lakebed deposit overlying a glacial till deposit, do not subdivide the lakebed deposit because adjacent samples have differing amounts of silt and clay. Realize before breaking down the soil profile that probably only two layers exist and variations are to be expected within each. Important variations such as average thickness of silt and clay varves can be noted adjacent to the visual description of the layer.

2. Remembering that the soil samples examined are only a minute portion of the soil underlying the site and must be considered in relation to not only adjacent samples, but also adjacent borings.

A few simple rules should be followed at this stage to properly interpret the available data:

1. Review the U.S.D.A. County Soil Map and determine major deposits expected at the site.

2. Examine the subsurface log containing standard penetration test results and the laboratory visual descriptions with accompanying moisture contents.

3. Personally review representative soil samples to check laboratory identification and to calibrate your interpretation with the laboratory technicians who performed the visual.

4. Establish rational mechanics for drawing the soil profile.

    a. Use a vertical scale of 1-inch equals 10 feet or 20 feet; generally, any smaller scale tends to squeeze data and prevent interpretation.

    b. Use a horizontal scale equal to the vertical, if possible, to simulate actual relationships. However, the total length should be kept within 36 inches to permit review in a single glance.

When the soil layer boundaries and descriptions have been established, determine the extent and details of laboratory testing. Consolidation and triaxial tests are expensive. Do not casually read the drillers' log and randomly select certain samples for testing. Plan the test program intelligently from the soil profile. Identify major soil deposits and assign appropriate tests for the design project under investigation.

The final soil profile is the foundation engineer's best interpretation of all available subsurface data. The final soil profile should include the average physical properties of the soil deposits, i.e., unit weight, shear strength, etc., in addition to a visual description of each deposit observed water level, and special items such as boulders or artesian pressure. Successful development of this subsurface profile will allow the foundation engineer to advance his design with confidence.

## 3.7 APPLE FREEWAY DESIGN EXAMPLE – BASIC SOIL PROPERTIES

In this chapter the process of establishing the basic soil properties based on visual description (logs), classification test (laboratory) and construction a soil profile are illustrated with reference to this Apple Freeway Example Design. The boring logs (Exhibit D of Chapter 2 Apple Freeway Example) and hypothetical moisture content tests data are used to illustrate how a preliminary soil profile is established for analysis and design.

**Given:** Boring logs and soil test data (Chapter 2 Apple Freeway Design Example)

**Required:** Determine preliminary soil profile

**Solution:**

**Step 1:** **Locate the borings in plan and elevation**

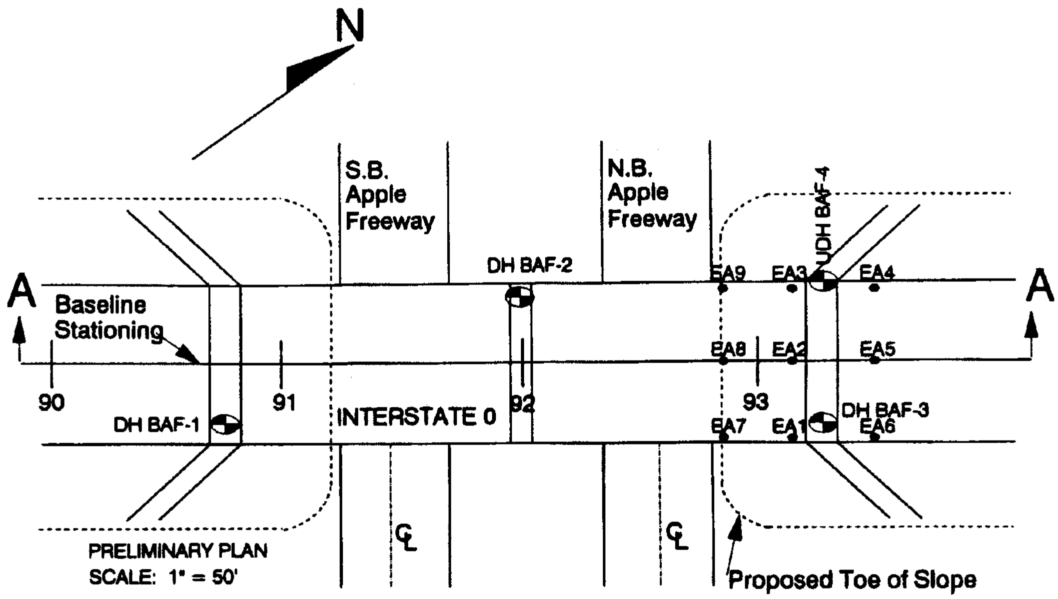
**Step 2:** **Plot the variation of field SPT value and classification test data (moisture content W) with depth.**

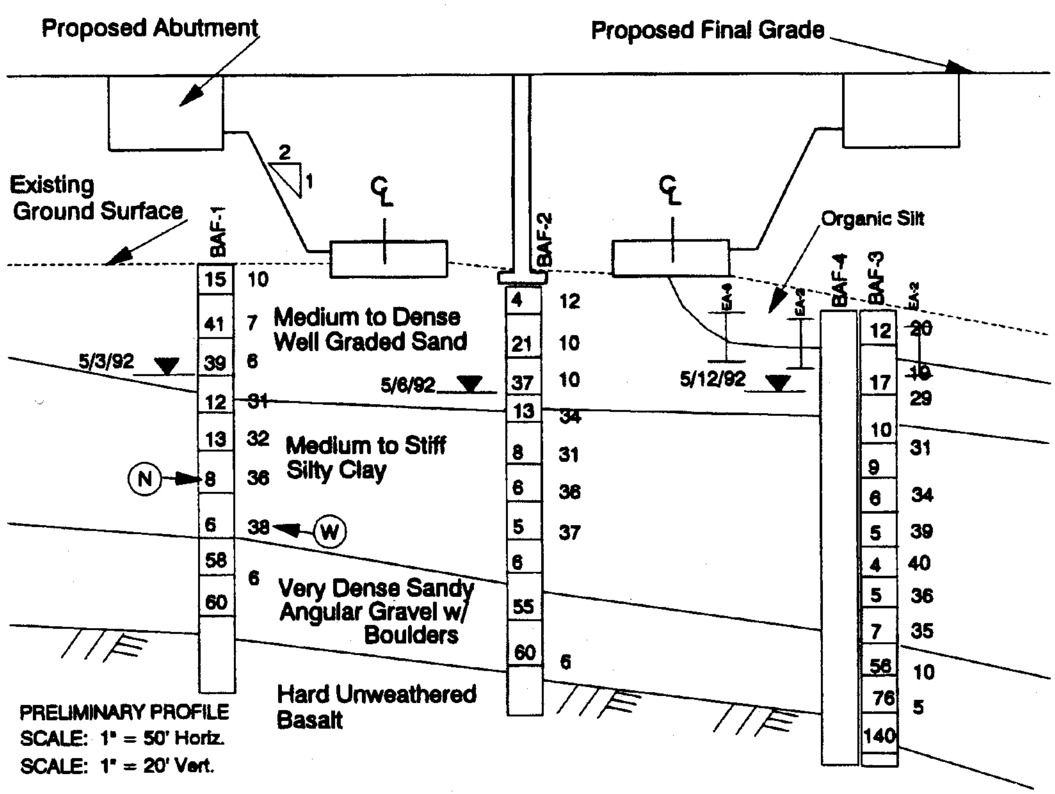
**Step 3:** **Plot the observed water levels in the borings and the date observed.**

**Step 4:** **Extrapolate between zones of similar properties based on site reconnaissance in formation, visual description and classification tests to establish preliminary soil profile.**

| | | |
|---|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings | |
| | **Basic Soil Properties** | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | $P_o$ Diagram<br>Test Request<br>Consolidation Results<br>Strength Results | |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze | |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains | |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge | |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement | |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation | |

Apple Freeway Design Example – Basic Soil Properties
Exhibit A




Apple Freeway Design Example – Basic Soil Properties
Exhibit B – Workshop Design Problem Preliminary Soil Profile

**Summary of the Basic Soil Properties Phase for Apple Freeway Design Problem**

- Visual Description

  Predominant soil types are sand, silty clay and sandy gravel.

- Classification Tests

  Moisture content and unit weight determined.

- Soil Profile

  Subsurface variation of soil layers and ground water estimated.

# CHAPTER 4.0
# LABORATORY TESTING FOR FOUNDATION DESIGN

Laboratory testing is an important element in foundation engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength testing. However, testing can be expensive and time consuming. The foundation engineer should recognize the project problems to be solved so as to optimize testing; particularly strength and consolidation testing.

However, before describing various soil test methods, the behavior of soil under load will be examined and common soil mechanics terms introduced. The following discussion only includes basic concepts of soil deformation behavior and only deals with saturated soils. The engineer must grasp these concepts to understand why particular types of soil testing are necessary to solve particular highway problems. The terms and symbols shown will be used throughout this manual. Basic soils textbooks should be consulted for detailed explanation of terms.

A sample of soil may be composed of soil grains, water and air. The soil grains are irregularly shaped solids which are in contact with other adjacent soil grains. The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the area between soil grains (voids or pores) and the amount of void space filled with water. Common terms associated with weight-volume relationships are shown in Table 4 – 1. Of particular note is the void ratio (e) which is a general indicator of the relative strength and compressibility of the soil sample, i.e., low void ratios generally indicate strong, incompressible soils, high void ratios may indicate weak, compressible soils.

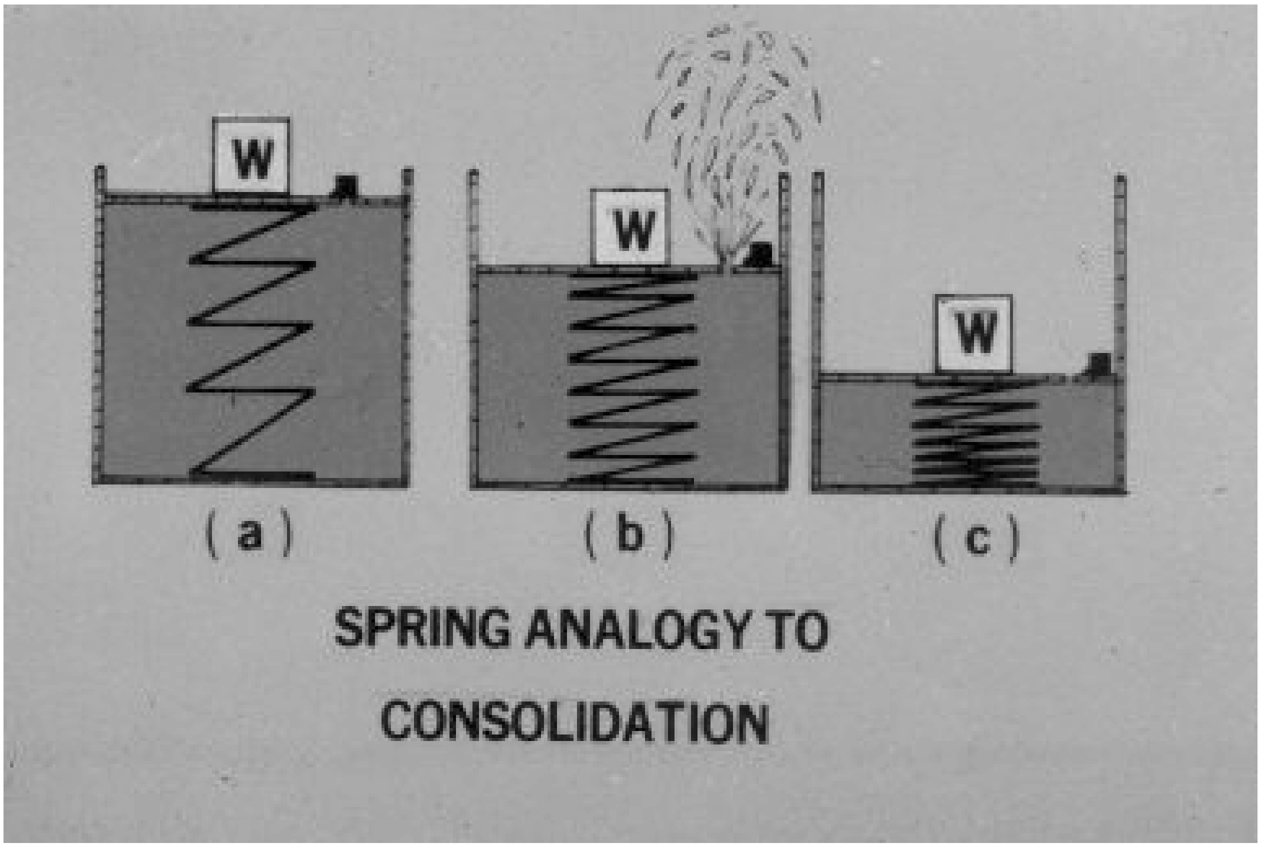
## TABLE 4 – 1
## WEIGHT VOLUME CHARACTERISTIC

| Property | Symbol | Units[1] | How To Obtained | Direct Applications |
|---|---|---|---|---|
| Moisture content | W | D | Directly from test AASHTO T93 | Classification and in volume-weight relations. |
| Unit weight | $\gamma$ | $FL^{-3}$ | Directly from test or from volume- weight relations AASHTO T38 | Classification and for pressure computations. |
| Porosity | n | D | Computed from volume-weight relations | Parameters used to represent relative volume of solids to total volume of soil. |
| Void ratio | e | D | Computed from volume-weight relations. | |
| Specific gravity | $G_S$ | D | Directly from AASHTO T100 | Volume computations. |

UNITS[1]: F=Force or weight; L=Length; T=Time; D=Dimensionless

When a load is applied to a soil sample, the deformation which occurs will depend on the grain to grain contacts (intergranular forces) and the amount of water in the voids (pore water). If no pore water exists, the sample deformation will be due to sliding between soil grains and deformation of individual, soil grains. Experience has shown that rearrangement of soil grains due to sliding accounts for the most deformation. Adequate deformation is required to increase the grain contact areas to take the applied load. As the amount of pore water in the void increases, the pressure exerted on soil grains will increase and reduce the intergranular contact forces. In fact, tiny clay particles may be forced completely apart by water in the pore space.

Deformation of a saturated soil is more complicated than dry soil as water molecules, which fill the voids, must be squeezed out of the sample before readjustment of soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil sample is quickly increased, the increase is carried entirely by the pore water until drainage begins. Then more and more load is gradually transferred to the soil grains until the excess pore pressure has dissipated and the soil grains readjust to a denser configuration. This process is called consolidation and results in a higher unit weight and a decreased void ratio.




## 4.1 PRINCIPLES OF EFFECTIVE STRESS

The consolidation process demonstrates the very important principle of effective stress, which will be used throughout this manual. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and pore pressure (neutral stress). As the pore water has no strength and is incompressible, only the intergranular stress is effective in resisting shear or limiting compression of the soil sample. Therefore, the intergranular contact stress is called the effective stress. When pore water drains from soil during consolidation, the area of contact between soil grains increases, which increases the level of effective stress. Stage construction of embankments is used to permit increase of effective stress in the foundation soil before subsequent fill load is added. In such operations effective stress increase is frequently monitored with piezometers to insure the next stage of embankment can be safely placed. Simply stated, the principle of effective stress states that the total stress on any plane within a soil mass is equal to the sum of the effective stress and the pore pressure.

In general, soil deposits below the water table will be considered saturated and the ambient pore pressure at any depth, may be computed by multiplying the unit weight of water by the height of water above that depth. The total stress at that depth may be found by multiplying the total unit weight of the soil by the depth. The effective stress is the total stress minus the pore pressure.

## 4.2 OVERBURDEN PRESSURE

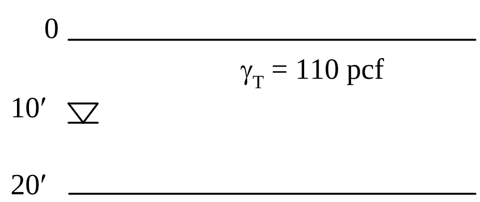
The laboratory testing required to solve soil-related problems involves simulating conditions naturally existing in the ground. Soils existing a distance below ground are affected by the weight of the soil above that depth. The influence of this weight, known generally as overburden pressure, causes a state of stress to exist which is unique at that depth, for that soil. When a soil sample is removed from the ground, that state of stress is relieved as all confinement of the sample has been removed. In testing, it is important to reestablish the in situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. As previously mentioned, the effective stress (grain to grain contact) is the controlling factor in shear and consolidation.

The test stresses are estimated from either the total or effective overburden pressure. The engineers' first task is determining the total and effective overburden pressure variation with depth. This relatively simple job involves determining the average total unit weight for each soil layer in the soil profile, and determining the depth of the water table. Unit weight may be accurately determined from density tests on undisturbed samples or estimated from standard penetration values and soil visuals. The water table depth, which is standardly recorded on boring logs, can be used to compute the pore pressure at any depth. The total overburden pressure ($P_T$) is found by multiplying the total unit weights of each soil layer by the layer thickness and continuously summing the results with depth. The effective overburden pressure ($P_o$) at any depth is determined by accumulating the weights of all layers above that depth with consideration of the water level conditions at the site as follows:

1. Soils <u>above</u> the water table - multiply the total unit weight by the thickness of each respective soil layer above the desired depth, ie, $P_o = P_T$.

2. Soils <u>below</u> the water table - subtract pore pressure ($\mu$) from $P_T$ or reduce the total unit weights by the weight of water (62.4 pcf), ie, use effective unit weights $\gamma_b$ and multiply by the thickness of each respective soil layer between the water table and the desired depth $P_o = P_T$ - ($\gamma_w$ x depth), or $\gamma_b$ x depth.

**Example 4-1:** Find $P_0$ at 20 feet below ground in a sand deposit with a total unit weight of 110 pcf and the water table 10 feet below ground. Plot $P_T$ and $P_0$ verses depth from 0′ – 20′.




Solution:

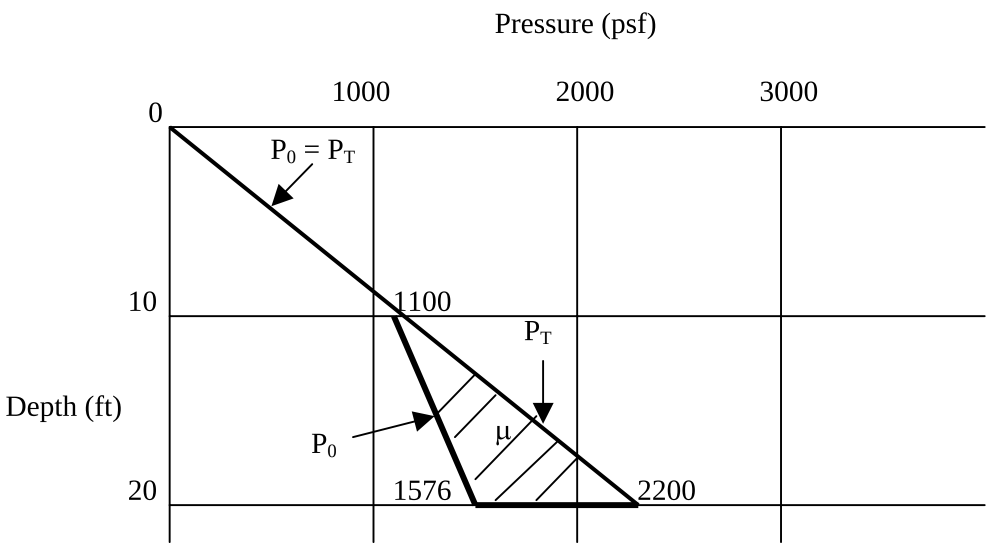
$$P_0 = P_T - \mu$$

$$P_T \text{ @ } 10' = P_0 \text{ @ } 10' = 10' \times 110 \text{ pcf} = 1100 \text{ psf}$$

$P_T$ @ 20′ = $P_T$ @ 10′ + (10′ × 110 pcf) = 2200 psf

$\mu$ @ 20′ = 10′ × 62.4 pcf = 624 psf

$P_0$ @ 20′ = $P_T$ @ 20′ - $\mu$ @ 20′ = 2200 – 624 = 1576 psf




**Pressure Diagram**

A PLOT OF EFFECTIVE OVERBURDEN PRESSURE VERSUS DEPTH IS CALLED A $\underline{P_0}$ DIAGRAM AND IS USED THROUGHOUT ALL ASPECTS OF FOUNDATION TESTING AND ANALYSIS.

### 4.3 USE OF ATTERBERG LIMITS

The following are the more important uses of Atterberg limits in determining engineering properties of soils:

1. Help identify and classify the soil.

2. PI (plasticity index) is an indicator of soil compressibility and potential for volume change. Estimate compression index ($C_c$) for normally consolidated and low sensitivity clay in preliminary design using:

$$C_c \cong 0.009\ (LL-10)$$

3. PL (plastic limit) can indicate if clay has been preconsolidated. Most soils are deposited at or near their liquid limit. If the in situ natural water content (W) is near the plastic limit (PL), then the soil is probably preconsolidated. Some stress has been applied in the past to squeeze that water out.

4. Clay may also be assumed to be preconsolidated if the liquidity index (LI), which is the (moisture content minus plastic limit) divided by plastic index is less than 0.7.

Atterberg limit formation from a specific site is frequently plotted on the “A-line” diagram to assess basic soil properties.

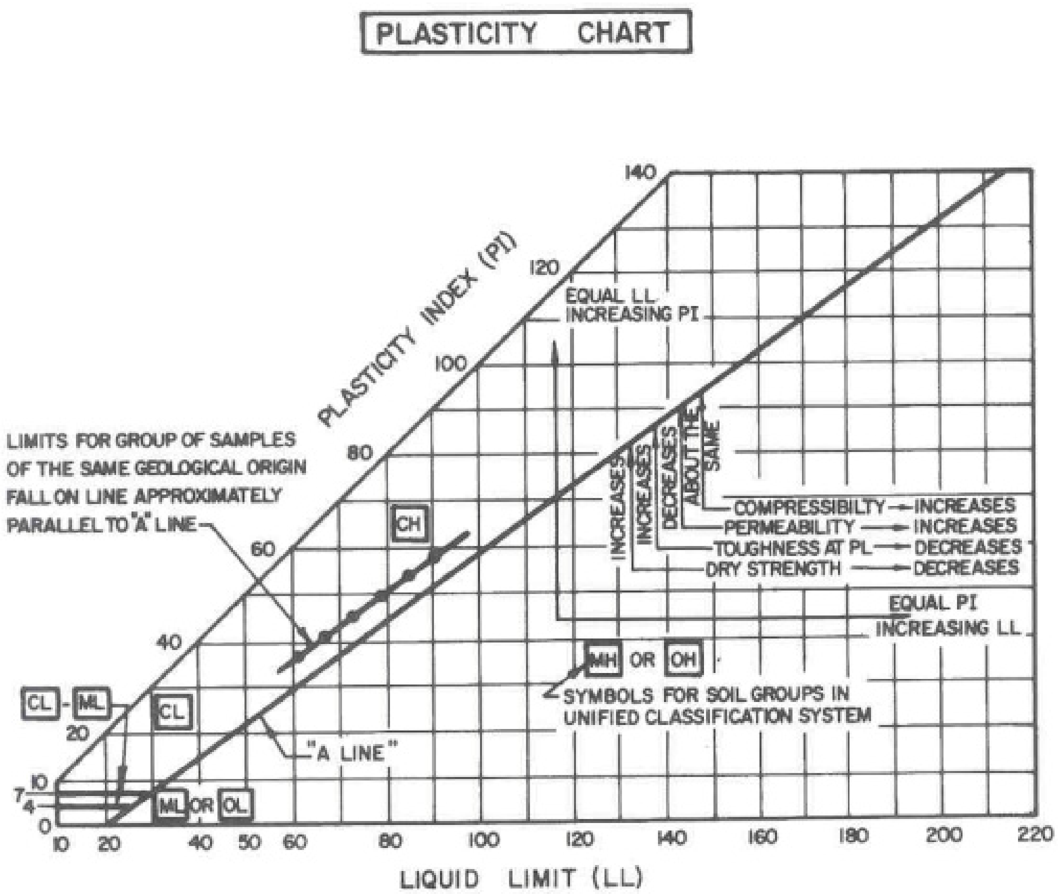



Figure 4-1 Plasticity (A-Line) Chart

The value of this simple procedure is great as noted by Arthur Casagrande in his “Discussion of Requirements for the Practice of Applied Soil Mechanics”, in the first Pan American Conference on Soil Mechanics and foundation Engineering, September 1959.

> I consider it essential that an experienced soils engineer should be able to judge the position of soils, from his territory, on a plasticity chart merely on the basis of his visual and manual examination of the soils. And more than that, the plasticity chart should be for him like a map of the world. At least for certain areas of the chart, that are significant for his activities, he should be well familiar. The position of soils within these areas should quickly convey to him a picture of the significant engineering properties that he should expect.

## 4.4 EFFECTS OF TEMPERATURE EXTREMES

As soil samples contain a percentage of water, exposure to temperature extremes after the sample has been removed from the ground will produce permanent undesirable changes in the soil's engineering properties. The two primary causes of temperature extremes are poor transport and storage of samples prior to testing. Freezing and thawing cycles will destroy soil structure which results in reduced shear strength and increased compressibility test results. Heating will dry soil samples and result in artificially high strength and low compressibility test results. Undisturbed samples should be tested as soon as possible after extraction and only stored in temperature/humidity controlled areas.

The life of properly stored samples varies but recommended practical maximums are two months for sensitive soils (sensitivity > 4). Watch for telltale warning signs of sample shrinkage or oxidation when extruding older samples.

## 4.5 LABORATORY TESTING GUIDELINES

An experienced geotechnical engineer can only decide certain considerations regarding laboratory testing, such as when, how much, and what type. The following guidelines are presented.

1. Perform a laboratory visual identification on all soil samples extracted from the borings.

2. Perform a moisture content analysis on all samples (cohesionless samples may be excluded if the number of samples becomes great). Classification tests may be performed on selected samples as requested by the designer.

*3. Perform an adequate number of consolidation tests on cohesive soil samples to determine variation of preconsolidation pressure with depth. Estimate one test every 5 feet for the top 20 feet of a cohesive deposit and one test each 10 feet thereafter. Normally consolidated soils may be tested at intervals of 10 feet throughout.

*4. Perform shear strength tests in each definable soil deposit. Each cohesive deposit should have at least one 3-point consolidated undrained test with or without pore pressure measurements at 10 to 15-foot intervals. The 3-point tests should be consolidated at equal pressure increments between existing effective overburden pressure, and the final effective pressure developed under the embankment loading. Unconsolidated, undrained tests may be performed on remaining undisturbed samples at confining pressures above total overburden. All unconsolidated undrained tests should only be done on samples extruded directly from the sampling tube and tested untrimmed at full diameter.

* Note that these tests may be costly and time consuming. An experienced geotechnical engineer must schedule or review all testing requests before implementation. Laboratory testing is not required on many routine projects.

A list of common soil properties routinely used in design, which can be determined by laboratory testing, is presented in Table 4-3.

# TABLE 4 – 2
# COMMON SOIL PROPERTIES

Many other soil properties are determined by testing and routinely used in design. A list of some common properties is presented below.

| PROPERTY | SYMBOL | DIM. | HOW OBTAINED | USE |
|---|---|---|---|---|
| Gradation characteristics<br>Effective diameter... | $D_{10}$ | L | AASHTO T88. From<br>Grain-size curve | Classification, estimating permeability and unit weight, filter design, grout selection, & evaluating potential frost heave |
| Percent grain size | $D_{30}$, $D_{60}$, $D_{85}$ | L | From grain size curve | |
| Coefficient of uniformity | $C_u$ | D | $D_{60}$ / $D_{10}$ | |
| Coefficient of curvature | Cz | D | $(D_{30})^2$ / ($D_{10}$ x $D_{60}$) | |
| Clay size fraction | … | D | From grain-size curve, % finer than 0.002 mm | Classification |
| Consolidation characteristics:<br>Coefficient of compressibility | $a_v$ | $L^2$ $F^{-1}$ | Determined from arith. e vs. p curve | Computation of ultimate settlement or swell in consolidation analysis |
| Coefficient of volume Compressibility | $m_v$ | $L^2$ $F^{-1}$ | $a_v$ / 1 + e | |
| Compression index | $C_c$ | D | Determined from semilog e vs p curve.<br>AASHTO T216 | |
| Recompression index | $C_r$ | D | | |
| Swelling index | $C_s$ | D | | |
| Coefficient of secondary compression | $C_\alpha$ | D | Determined from semilog time-consolidation curve | |
| Coefficient of consolidation | $C_v$ | $L^2$ $T^{-1}$ | | Computation of time rate of settlement |
| Preconsolidation pressure | $P_c$ | $FL^{-2}$ | Estimated from semilog e vs p curve | Consolidation analysis |
| Shear strength characteristics:<br>Angle of internal friction | $\phi$ | D | Determined from Mohr envelope for total normal stress.<br>AASHTO T234 | Analysis of stability and load carrying capacity of foundations. |
| Cohesion intercept | c | $FL^{-2}$ | | |
| Angle of internal friction | $\phi'$ | D | Determined from Mohr envelope for effective normal stress | |
| Cohesion intercept | c' | $FL^{-2}$ | | |
| Unconfined compressive strength | $q_u$ | $FL^{-2}$ | Directly from test<br>AASHTO T208 | |
| Shear strength | s | $FL^{-2}$ | | |
| Sensitivity | $S_t$ | D | $q_u$(undisturbed) ÷$q_u$(remolded) | Estimating effect of disturbance in driving piles |
| Modulus of elasticity | $E_s$ | $FL^{-2}$ | Determined from stress - strain curve | Computation of elastic settlement or rebound |
| Characteristics of compacted samples:<br>Maximum dry unit weight | $\gamma_{max}$ | $FL^{-3}$ | Determined from moisture-density curve<br>AASHTO T99<br>AASHTO T180 | Compaction control and computation of weights and forces in stability analysis |
| Optimum moisture content | OMC | D | | |
| Relative density | $D_d$ | D | Determined from results of max and min density tests | Compaction control |
| California bearing ratio | CBR | D | Directly from test<br>AASHTO T193 | Pavement Design |

Reference: NAVFAC DM-7     F=Force, L=Length, T=Time, D=Dimensionles

## 4.6 PROCESS OF CONSOLIDATION

Consolidation is a decrease in the volume of a soil due to static loading or vibrational forces applied to the soil mass. In highway design, static loading is represented by the permanent load placed on the soil by embankments and structures. As most compressible foundation soils are below the water table, all the voids are filled with water. An applied load will cause soil grains to readjust to a more compact position in order to carry the load. This readjustment cannot take place until the water, which is incompressible, escapes from the voids. In impervious soils which have small voids, water will travel very slowly.

Years may be needed for the water to drain away from the loaded soil area so the settlement can go to completion. The general consolidation characteristics of various major soil types are used to determine if a highway settlement problem may be anticipated. The following three groupings portray these characteristics:

| | | |
|---|---|---|
| 1. | Gravels, sands, and non-plastic silts (granular soils) | Relatively incompressible. Will consolidate immediately under load when fill is placed. These soils do not present embankment settlement problems |
| 2. | Plastic silt-clay mixtures (cohesive soils) | Soft silts and clays are more compressible than stiff silts and clays. Settlement may continue long after construction. |
| 3. | Organic soils | Very compressible, and settlement of large magnitude will continue for years. |

Consolidation occurs in three stages; initial, primary, and secondary. Initial (elastic) compression occurs simultaneously with application of load and is usually quite small. It is due primarily to compression of air and gas in the soil voids. Primary compression normally is the largest part of the total compression that will occur. Water has to be squeezed out of the soil voids for primary consolidation to take place. Therefore, the amount of primary consolidation will depend on the initial void ratio of the soil. The greater the initial void ratio, the more water that can be squeezed out, and the greater the primary consolidation. The rate at which primary consolidation occurs is dependent on the rate at which the water is squeezed out of the soil voids. Secondary compression (creep) occurs after primary consolidation is complete. Secondary compression is not dependent on water being squeezed out of the soil. Secondary compression can occur under constant load. It is caused by the soil particles reorienting or deforming under constant load.

Initial compression accounts for the major portion of consolidation in granular soils. Primary compression accounts for the major portion of consolidation in cohesive soils. Primary and secondary compression both contribute significantly to organic soil consolidation.

Some natural deposits of cohesive soils have undergone heavy compression in geologic history (due to the weight of glaciers, due to the weight of overlying soil that has been eroded off, or due to desiccation) and are therefore relatively incompressible Such soils are called preconsolidated or overconsolidated and have been subjected to greater stresses in the past than at present. This is important because these soils can be reloaded (such as by weight of an embankment or bridge footing) and will not settle appreciably until the reapplied load exceeds the preconsolidation load. Cohesive deposits, which have never been subjected to previous compression, are called normally consolidated. This means the soil has never been subjected to an overburden stress any greater than the stress existing at the present time.

### 4.6.1 Consolidation Testing

In order to predict the amount of consolidation in cohesive and organic soils, adequate testing must be performed. The undisturbed soil sample to be tested should be obtained in the field with a thin wall tube sampler. The designer should instruct the laboratory as to how many tests should be performed and modifications to standard procedures. The test request (Figure 4 – 2) should include the following information.

1. Clear designation of which samples are to be tested. Usually consolidation testing is done in close intervals (5’) near the layer top and at wider intervals (10’) at greater depths.

2. Loading time increments should be specified to optimize production. Minimal time increments may be used for adding test loads up to one load before $P_o$. Thereafter, three hour-increments may be used for soils with a moisture content less than 50 percent while twenty-four hour - increments are needed for highly organic soils and very plastic clays. Longer load durations may be needed in highly organic soils to define the coefficient of secondary compression accurately.

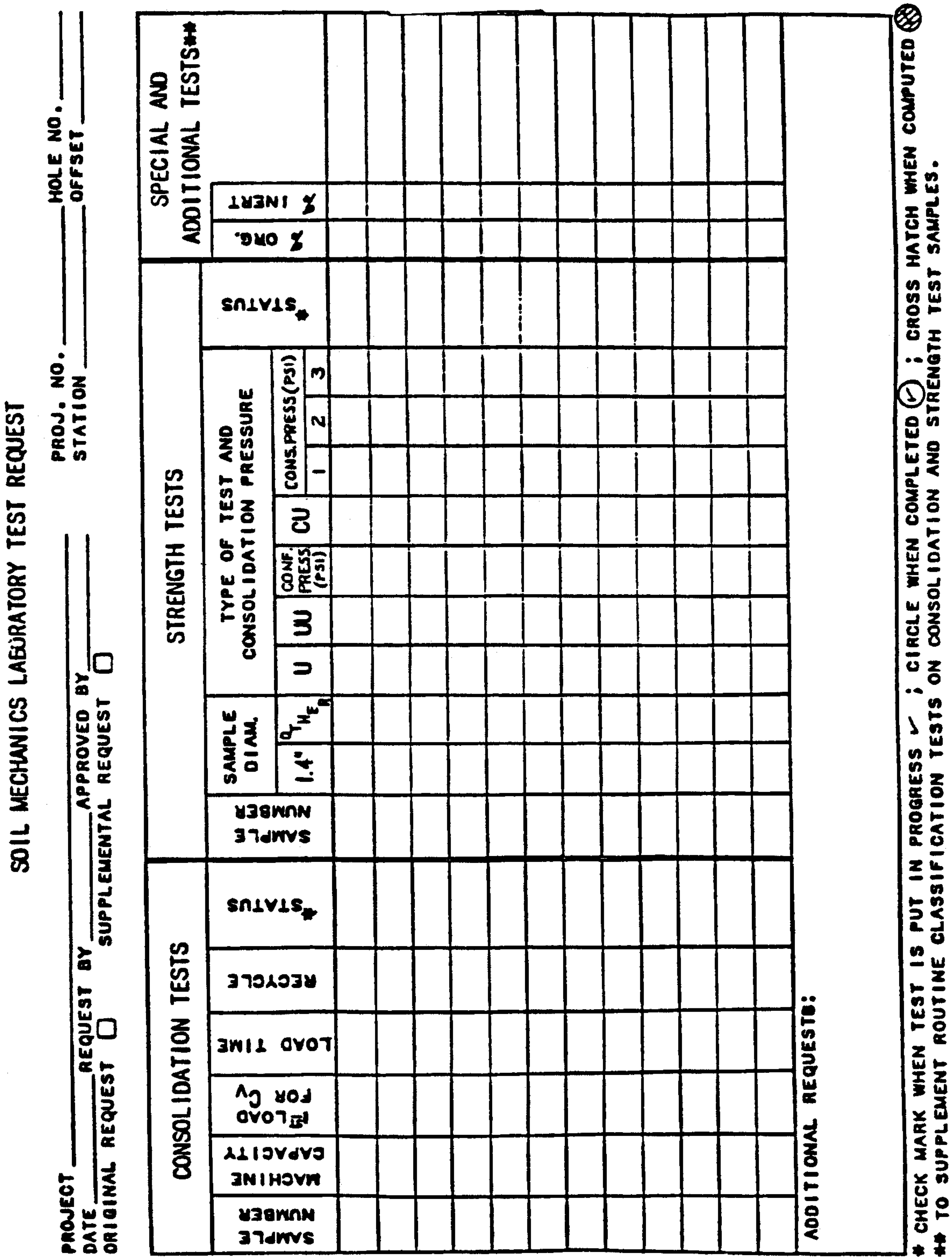
3. The load range where the coefficient of consolidation is to be computed should be specified. Generally, values are computed starting at the load below effective overburden pressure at the depth of the tube sample.

4. The recycle loads, (if needed for very accurate settlement prediction) should be specified to start at one load beyond the preconsolidation pressure and return to one load below effective overburden pressure before reloading to the requested maximum test load.

The consolidation results are generally presented graphically as shown in Figure 4 – 3.

The pressure-void ratio plot is used to find the preconsolidation pressure and other values pertaining to the compression of the soil sample. Both the arithmetic and semi log pressure-void ratio plots have been shown although the semi log plot is recommended and will be used in subsequent sections of this manual. On the semi log pressure-void ratio plot (e vs. log P), the engineer can readily see the sharp break in the curve at $P_c$ which indicates compression will increase rapidly for additional increases in load beyond the preconsolidation pressure. The semi log time-compression curve ($t_{50}$) is used to find the secondary compression and the time rate of consolidation for the soil sample.

Some geotechnical engineers prefer to use a plot of percent strain versus log of pressure is used instead of the e vs. log P plot. In this case the interpreted values of compression and recompression indices reflect the relationship between strain and void ratio, i.e., strain = $\Delta e/(1+e_0)$. To convert the strain based indices to the void ratio based indices ($C_c$ and $C_r$) multiply the strain based values by $1 + e_o$. Void ratio based values (e vs. log P) will be used in the remainder of this book.



SOIL MECHANICS LABORATORY TEST REQUEST

PROJECT ____________ PROJ. NO. ________ HOLE NO. ________
DATE ________ REQUEST BY ________ APPROVED BY ________ STATION ________ OFFSET ________
ORIGINAL REQUEST ☐ SUPPLEMENTAL REQUEST ☐

| CONSOLIDATION TESTS | | | | | | STRENGTH TESTS | | | | | | | | | | | | SPECIAL AND ADDITIONAL TESTS** | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SAMPLE NUMBER | MACHINE CAPACITY | 1st LOAD FOR $C_V$ | LOAD TIME | RECYCLE | *STATUS | SAMPLE NUMBER | SAMPLE DIAM. | | TYPE OF TEST AND CONSOLIDATION PRESSURE | | | | | | | *STATUS | % ORG. | % INERT | |
| | | | | | | | 1.4" | OTHER | U | UU | CONF. PRESS (PSI) | CU | CONS. PRESS (PSI) | | | | | | |
| | | | | | | | | | | | | | 1 | 2 | 3 | | | | |
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ADDITIONAL REQUESTS:

* CHECK MARK WHEN TEST IS PUT IN PROGRESS ✓ ; CIRCLE WHEN COMPLETED ; CROSS HATCH WHEN COMPUTED
** TO SUPPLEMENT ROUTINE CLASSIFICATION TESTS ON CONSOLIDATION AND STRENGTH TEST SAMPLES.

Figure 4-2: Laboratory Test Request Form

Analyze consolidation test data to determine:

1. Preconsolidation Pressure ($P_c$)

   The maximum pressure to which a soil has been loaded in the past will have a major influence on the amount of settlement to be expected under a proposed loading. In fact, 10 times more settlement may occur in an unconsolidated soil than a preconsolidated soil for equal load increments. These preconsolidation values should be carefully established for the entire depth of the cohesive deposit under consideration. Normally, a maximum and minimum value of $P_c$ will be established and plotted as a range with depth.

2. Compression Index ($C_c$) and Recompression Index ($C_r$)

   The slope of virgin compression and recompression portions of the e vs. log P curve is respectively $C_c$ and $C_r$. In general, $C_c$ is approximately 10 times greater than $C_r$. The point where lines drawn tangent to the slopes intersect is the minimum preconsolidation pressure. The $C_c$ and $C_r$ values are respectively estimated by dividing the soil moisture content by 100 and 1000. As their names imply, the values are a direct measure of soil compression.

3. Initial Void Ratio ($e_o$)

   The value of $e_o$ is determined prior to application of load. The value $e_o$ is used in settlement computations to determine settlement magnitude.

4. Coefficient of Consolidation ($C_v$)

   This parameter is an indicator of the rate of drainage during consolidation; or in the case of pile driving an indicator of the time required for remolded soil to gain strength and reconsolidate around the pile. The value may be determined by $t_{50}$ (as previously shown) or the $t_{90}$ (square root of time) method which is described in many soil textbooks. A plot of $C_v$ versus log P will show a sharp decrease at the preconsolidation pressure.

5. Coefficient of Secondary Compression ($C_\alpha$)

   Of great importance in organic materials, this value may account for the majority of strata consolidation. The $C_\alpha$ value is determined from the $t_{50}$ semi log time-compression curve.

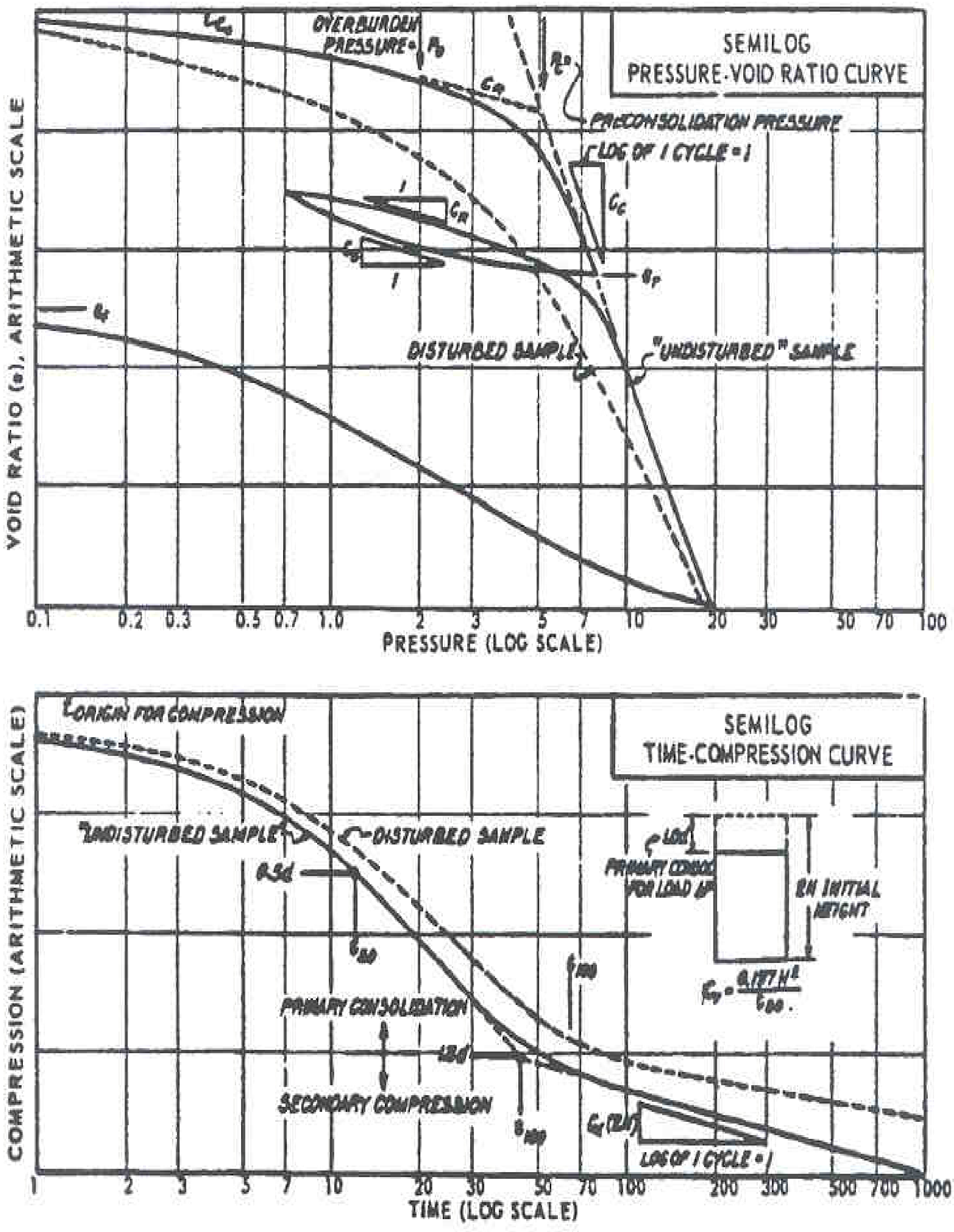
6. Effects of Sample Disturbance on Consolidation Test Results

   In previous pages the subject of undisturbed soil sampling was addressed. The importance of obtaining good quality samples was stressed. The influence of disturbance on consolidation test values is as follows and as shown on Figure 4 – 3.

   a. Eliminates the distinct break in the e vs. log P curve at the preconsolidation pressure.

   b. Lowers the estimate of preconsolidation pressure and the measured compression index.

   c. Decreases measured $C_v$ values and eliminates the sharp break in e vs. log P at the preconsolidation pressure.




*Reference NAVFAC DM-7

Figure 4 – 3: Consolidation Test Relationships

d. Increases the recompression index.

e. Decreases the secondary compression coefficient.

The general effects of disturbance are under or over-prediction of the magnitude of expected settlement and over-prediction of the time for its occurrence.

The general importance of the consolidation test results applied to design are shown below. The test results may be applied to project design after a series of tests have been completed to represent the total depth of the soil deposit. The two most important predictions are:

1. The amount of settlement which may be determined by analyzing the test sample compression between the overburden pressure and the final pressure induced by the highway load at various depths. The compression may vary dramatically depending on the maximum past pressure to which the soil has been loaded.

2. The time for settlement may be estimated from the results of the compression versus time plots at loads between the overburden pressure and final pressure induced by the highway load. The important factors in the time-settlement relationship are:

   a. Time required is proportional to the square of the longest distance required for water to drain from the deposit. This distance is the thickness of the layer if all water drains only vertically to the surface, and one-half the layer thickness if more permeable soils also exist below the layer.

   b. Time required for consolidation varies inversely as the coefficient of consolidation.

   c. Rate of settlement decreases as time increases.

## 4.7 SOIL STRENGTH

The most important property of soils is strength. Slopes of all kinds, including hills, river banks, and man-made cuts and fills, stay in place only because of the strength of the material of which they are composed. Knowledge of soil strength is important for the design of structure foundations, embankments, retaining walls, pavements, and cuts.

Basic concepts indicate a soil can derive strength from two sources; friction between particles and cohesion between particles.

1. Cohesionless soils, such as gravel, sand, and silt, derive strength from friction between particles.

2. Cohesive soils, composed mainly of clay, derive strength from the attraction, or bond, between particles.

3. Mixtures of cohesionless and cohesive soils derive strength from both friction between particles and cohesion.

The frictional resistance between soil particles is dependent on the overburden pressure above the particles and the angle of internal friction between the particles. The total available shear strength (frictional resistance) is equal to the normal force times the tangent of $\phi$ (tangent of $\phi$ is equal to the coefficient of friction between the soil particles). The equation for frictional resistance is commonly written: $S = N \tan \phi$. A pile of dry sand will have a maximum angle of repose of about $30^o$. This is approximately equal to the friction angle between soil particles.

The coefficient of friction between individual particles depends on both their mineral hardness and the surface roughness. However, the measured friction angle of a soil sample or deposit will also depend on the density of the mass caused by interlocking of particles. Angular particles, which interlock better than round particles, are specified for base courses and flexible pavement due to their higher strength. Care must be taken in estimating the coefficient of friction between dissimilar materials such as pile-soil systems as the interlock contribution is not mobilized.

The concept of cohesive strength is more difficult to explain as the cohesion is dependent on nebulous quantities such as the ionic bond between soil mineral grains. However, the practical aspects are easily understood in the relation to granular soils. Dry granular soils are unable to stand at slopes steeper than their angle of repose. However, clay deposits can be cut vertically to some limiting depth. Clay particles maintain their position in vertical slopes due to attractive forces between adjacent clay particles. This attractive force is commonly called the cohesion of the clay. The magnitude of the cohesion is dependent on the distance between individual clay mineral particles. The greater the separation, the lower the attractive force, and the smaller the cohesion;the closer the particles, the higher the cohesion. Separation of adjacent clay particles is maintained by water molecules which fill the void spaces between particles. As water is squeezed out due to external applied loads, separation decreases and cohesion increases. A unique relationship exists between the shear strength and water content of a clay. Great importance is associated with developing a plot of shear strength versus moisture content for major projects. Then moisture content variations can be used to assess shear strength variations in drill holes where undisturbed samples are not extracted.

The time required for water to be squeezed out from between soil particles varies generally with the size of the particles. The shear strength of granular soil increases immediately as the load increases. The strength of a pure cohesive soil increases very slowly after load is applied since consolidation is required for strength gain. Therefore, placement of highway embankments on cohesive soils must be controlled to prevent the applied load exceeding the initial soil cohesive strength.

For practical purposes most cohesive clay deposits contain some non-cohesive silt or sand. Hence under an increased load some increase in soil strength can be expected. The shear strength of any soil is commonly denoted as:

Shear Strength (S) = Cohesion (C) + Normal Force (N) x Tangent of Friction Angle ($\phi$)

### 4.7.1 Strength Testing

The majority of strength tests are conducted on cohesive soils, as obtaining undisturbed samples of non-cohesive soils is difficult. Strength tests on cohesive soils are conducted on high quality undisturbed samples obtained from thin wall tubes. The number and type of test must be selected by the designer to suit the project conditions. For each test the designer should clearly indicate the consolidation or confining pressure to be used. These pressures are determined from the $P_o$ diagram for each specific project. The range usually extends from the effective overburden pressure to the pressure induced by highway loading. The program objective should be to establish a profile of soil strength with depth. Soil strength parameters are frequently expressed as a ratio of shear strength over the effective overburden pressure ($S/P_o$).

The most common soil strength tests are as follows:

1. The Unconfined Compression Test is the simplest and quickest laboratory method used to measure the shear strength of a cohesive soil. Test results, especially with increasing depth, are conservative and misleading due to the release of confining stress when the sample is removed from below ground and tested.

2. The Triaxial Compression Test is a strength test where the sample is subjected to confining pressures similar to those which existed in the ground before sampling. In general triaxial tests may be done on soil samples which have either been consolidated in the lab to the effective overburden pressure before testing or left unconsolidated and tested at total overburden pressure. In either case, the tests try to produce the in situ effective stress condition. Unconsolidated tests must be done soon after sampling to insure no changes have occurred in the amount of pore water in the sample. The consolidated triaxial compression test duplicates as accurately as possible the sample's conditions in the ground and gives an accurate indication of shear strength. A series of tests on samples consolidated under various confining pressures may be run in order to determine the amount of strength increase with consolidation under embankment loads. The triaxial test procedure may be varied to account for short term (undrained) load application or long term (drained) load application.

3. The Vane Shear Test is a field test made in conjunction with drill hole explorations in soft clays. This is a test for determining shear strength rapidly without laboratory testing. A post-test soil sample should be extracted from the test depth to permit correlation of strength with other physical properties. The vane test strength results are accurate in soft silts and clays. Miniature laboratory vane tests are not as dependable due to sample confinement in the tube, size effects, and sampling disturbance.

4. The Direct Shear Test is a relatively simple test used to measure the shear strength of fine granular soils. This test is not recommended for silts and clays as test sample drainage cannot be controlled during the test. Sophisticated direct simple shear testing is appropriate for fine grained soils but the necessary equipment is only available at a few specialized soil labs.

### 4.7.2 Discussion of Shear Strength Testing

The shear strength of a soil is the maximum shear stress that the soil structure can resist before failure. Shear stresses are carried by the structure of soil grains as the water filling the pores has no shear strength. However, the shear strength of the soil structure is indirectly dependent on the pressure in the pore water which influences the N term in $S = C + N \tan \phi$. Foundation designers must consider the effects of expected construction operations on the subsoils when planning a test program. For example, when a highway embankment or structure footing is suddenly placed on a soft clay deposit, the pore water initially carries all the load and the available shear strength does not increase until drainage begins and the pore pressure decreases. In planning a test program for such a situation the designer would request unconsolidated undrained triaxial tests to determine the critical strength values, i.e., the initial shear strength before consolidation begins. Additional consolidated undrained or drained tests would also be used to determine the increase in shear strength as consolidation occurs and pore pressures dissipate. These results can be used to determine alternate methods of safely applying the loads, especially if the critical unconsolidated undrained strength is insufficient to sustain the proposed loading. Stage construction involves placement of an increment of load and a waiting period to allow strength gain so the soil deposit can safely support the next load increment.

### 4.7.3 Strength Test Results

Strength testing results are generally reported either as a shear strength (S) or in terms of cohesion (C) and friction angle ($\phi$). Certain tests produce results that are limited in application. The following summary is generally applicable to tests on saturated cohesive soils unless otherwise stated.

1. Unconfined Compression (U)

   This test is widely used as a quick economical means of obtaining the approximately in situ shear strength of cohesive soils at shallow depths. The test results are presented in the form of a stress-strain plot where the shear strength is computed to be the maximum compressive stress divided by two. In cohesive samples this shear strength should approximately equal the cohesion as the test is performed at atmospheric pressure, i.e., N equals 0 in $S = C + N \tan \phi$. The reliability of this test is particularly poor with increasing sample depth (below about 30') because the sample tends to swell after removal from tube. Swelling causes greater particle separation and reduced shear strength. Swelling can be minimized by testing as soon as possible after removal from the tube and at full diameter. This reduces disturbance and preserves natural moisture content.

2. Unconsolidated Undrained (UU) Triaxial

   This test is also dependent on the soil sample retaining its original structure until testing occurs. All UU tests should only be done on samples extruded directly from the sampling tube and tested untrimmed at full diameter. The results are the shear strength existing at that depth. Theoretically, the test confining pressure may be varied substantially above the usually applied total overburden pressure without changing the test results as all increases in N are carried by the pore water. Practically, slight increases in shear strength will be noted due to sample drying (nonsaturation) and disturbance of original structure. These increases should not be interpreted as shear strength gain with increasing load. The UU shear strength may be used in quick loading situations such as rapid construction of a highway embankment where all the load is applied before the deposit can consolidate and gain strength.

3. Consolidated Undrained (CU) Triaxial

   This test is generally performed on sets of three soil samples taken from the same tube. Each sample is consolidated to a different effective stress. The shear strength of each sample is determined and plotted on a Mohr diagram. The result is a line (called an envelope) which intercepts the Y-axis at the cohesion value and has an inclination measured from the X-axis equal to the friction angle. The undrained shear strength values may be estimated from the envelope for any loading within the range of test pressures.

4. Consolidated Drained (CD) Triaxial

   This test is interpreted similar to the CU test except the envelope will usually have a smaller cohesion (typical value of <100 psf) and a larger friction angle. The results are used to duplicate long term loadings when excess pore pressures do not develop. CU tests with pore pressure measurements are used in place of CD tests due to savings in testing time.

5. Direct Shear (DS)

This test is suitable for granular soils (and clays if proper equipment is used). Stress-strain curves are usually produced for at least three soil samples, each at a different test pressure. The stress-strain measurements are usually extended substantially beyond the peak to determine the residual shear stress, i.e., the stress which remains constant with increasing strain. The peak and residual shear strengths are plotted versus confining pressure to determine rate of increase of strength with applied load.

### 4.7.4 Comparison of Laboratory and Field Strengths

Laboratory soil samples are obtained from the ground by sampling from boreholes and sealing and transporting these samples to the laboratory. The degree of disturbance affecting the samples will vary according to the type of soil, sampling method and the skill of the driller. At best some disturbance will occur from the removal of in situ stresses during sampling and laboratory preparation for testing. In general, disturbance tends to reduce the shear strength obtained from unconfined or unconsolidated tests and increase the strength obtained from consolidated tests. There is, therefore, considerable attraction for measuring shear strength in the field, in situ. The vane shear test is the most commonly used field test for obtaining shear strength in soft to medium clays. As the test is performed rapidly, the strength measured is indicative of the undrained shear strength. In reviewing different types of field and lab testing in clays to determine the undrained-shear strength, the designer should expect the vane shear test to provide the most accurate value with U and UU tests yielding lower results and CU tests yielding slightly higher results.

### 4.7.5 Selection of Design Shear Strength

Frequently, on a large project the designer will receive a huge quantity of undrained shear strength test results from both the field and lab. This mountain of data must be concisely summarized to permit rational interpretation of results. The tests should be analyzed on a hole-by-hole basis. All tests from one hole should be reviewed and the existing undrained shear strengths selected. The results for each type of test should be plotted versus depth to determine the pattern of strength variation for each test type with depth and to assess the reliability of the data, i.e., a CU test result that is lower than the U test result at the same depth should be considered suspect. The general pattern of in situ shear strength results should be to increase with depth in a normally consolidated clay deposit. Clays which have been overconsolidated may only exhibit this increase at greater depths as the amount of preconsolidation increases shear strength in upper portions of the soil deposit.

## 4.8 PRACTICAL ASPECTS FOR LABORATORY TESTING

A poor understanding sometimes exists among geologists, structural engineers, and some foundation engineers about the type and amount of laboratory testing required for a structure foundation design. This weakness may render subsequent foundation design analyses useless. Organizations which have neither the proper testing facilities nor trained soils laboratory personnel may contract testing to private consultants. This solution can only be effective if the organization's foundation engineer can confidently request the necessary testing and review the results to select design values. A fair estimate of consultant testing costs may be obtained by assuming the following number of man-days (md) per test and multiplying by current costs; visual description of an SPT sample including moisture content (0.05 md), visual description of a tube sample including moisture content and unit weight (0.1 md), classification tests (0.7 md), undrained triaxial tests (0.9 md), drained triaxial tests (2.0 md), consolidation tests (2.0 md). These values include all work required to present a completed test result to the foundation designer.

Blanket consultant contracts "to perform testing necessary for design" usually result in unnecessarily large quantities of testing being performed, much of which does not apply to the project foundation problems. For example, if your multi-span structure is crossing a soft clay deposit underlain by sands, do not spend inordinate amounts of time and money to determine all strength and consolidation parameters of the soft clay layer at pier locations. Realize that the pile foundation will be designed using SPT values found in the underlying granular soils and that the only possible laboratory testing needed in the soft clay layer may be to estimate drag forces on the abutment piles. Also do not permit non-standard strength testing such as torvanes, penetrometers, etc. which are not covered by ASTM or AASHTO standards. Such devices should only be used as field index tests for consistency determination.

## 4.9 APPLE FREEWAY DESIGN EXAMPLE – LABORATORY TESTING

In this chapter the Apple Freeway Design Example is used to demonstrate the preparation of a $P_0$ diagram to prepare a laboratory test request for consolidation and strength tests. Typical consolidation and strength tests results are included at the end of the chapter.

**Given:** Preliminary soil profile and soil unit weights (determined in chapter 3)

**Required:** Prepare test request for consolidation and strength testing

**Solution:**

**Step 1:** **Construct $P_0$ diagram at boring UDH BAF – 4. The boring where the samples for strength and consolidation tests were obtained**

**Step 2:** **Based on the pressure at each depth ($P_0$) specify loads, test duration and loading pattern for consolidation test and the confining and consolidation pressure for the UU and Cu tests.**

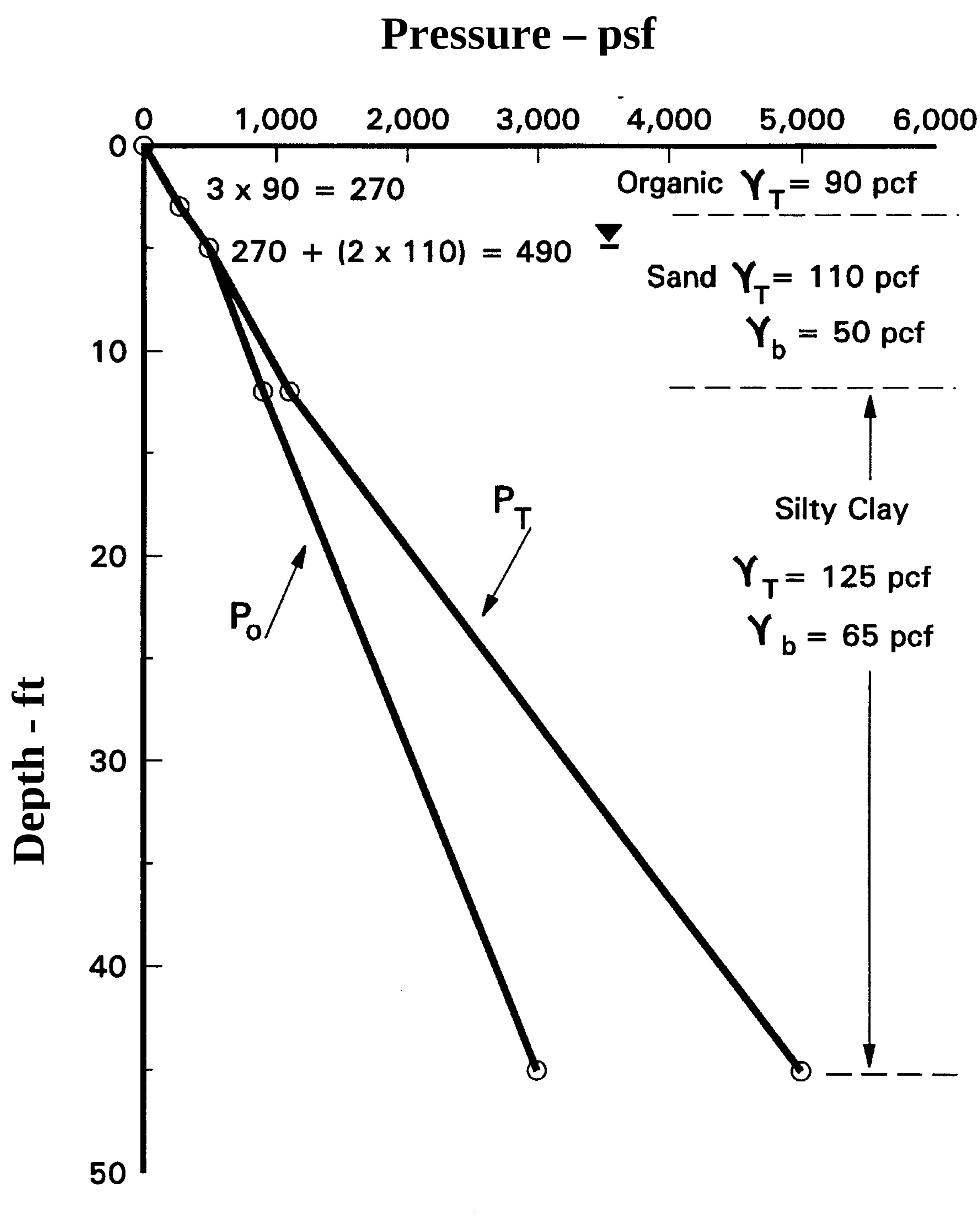
**Step 3:** **Use laboratory test results to obtain consolidation and strength parameters for design.**

| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| | **Laboratory Testing** |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

$P_o$ Diagram
Test Request
Consolidation Results
Strength Results

Apple Freeway Design Example – Laboratory Testing
Exhibit A

## Pressure Diagram ($P_0$ & $P_T$)



Soil Mechanics Laboratory Test Request

## SOIL MECHANICS LABORATORY TEST REQUEST

PROJECT *I-O APPLE FREEWAY* PROJ. NO. ______ HOLE NO. *BAF-4*
DATE *6-3-72* REQUEST BY ______ APPROVED BY ______ STATION *93+27* OFFSET *50' LT*
ORIGINAL REQUEST ☒ SUPPLEMENTAL REQUEST ☐

**CONSOLIDATION TESTS**

| SAMPLE NUMBER | MACHINE CAPACITY | 1st LOAD FOR $C_v$ | LOAD TIME | RECYCLE | *STATUS |
|---|---|---|---|---|---|
| T3 | 32K | 500 | 3 hr. | YES | |
| T4 | | 1000 | | YES | |
| T5 | | | | NO | |
| T6 | | | | YES | |
| T7 | | | | NO | |
| T9 | | | | NO | |

**STRENGTH TESTS** / **SPECIAL AND ADDITIONAL TESTS****

| SAMPLE NUMBER | SAMPLE DIAM. 1.4" | SAMPLE DIAM. OTHER | U | *UU | CONF. PRESS (PSI) | CU | CONS. PRESS (PSI) 1 ** | 2 | 3 *** | *STATUS | % ORG. | % INERT | SPECIAL AND ADDITIONAL TESTS** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| T4 | | FULL | | ✓ | 12 | | | | | | | | |
| T4 | ✓ | | | | | ✓ | 8 | 21 | 36 | | | | |
| T5 | | FULL | | ✓ | 17 | | | | | | | | |
| T5 | ✓ | | | | | ✓ | 10 | 24 | 38 | | | | |
| T6 | | FULL | | ✓ | 21 | | | | | | | | |
| T6 | ✓ | | | | | ✓ | 12 | 26 | 40 | | | | |
| T7 | | FULL | | ✓ | 26 | | | | | | | | |
| T7 | ✓ | | | | | ✓ | 15 | 29 | 43 | | | | |
| T9 | | FULL | | ✓ | 33 | | | | | | | | |
| T9 | ✓ | | | | | ✓ | 19 | 33 | 47 | | | | |
| | | | | * $P_{TOTAL}$ | | | ** $P_o$ | | *** $P_f$ | | | | |

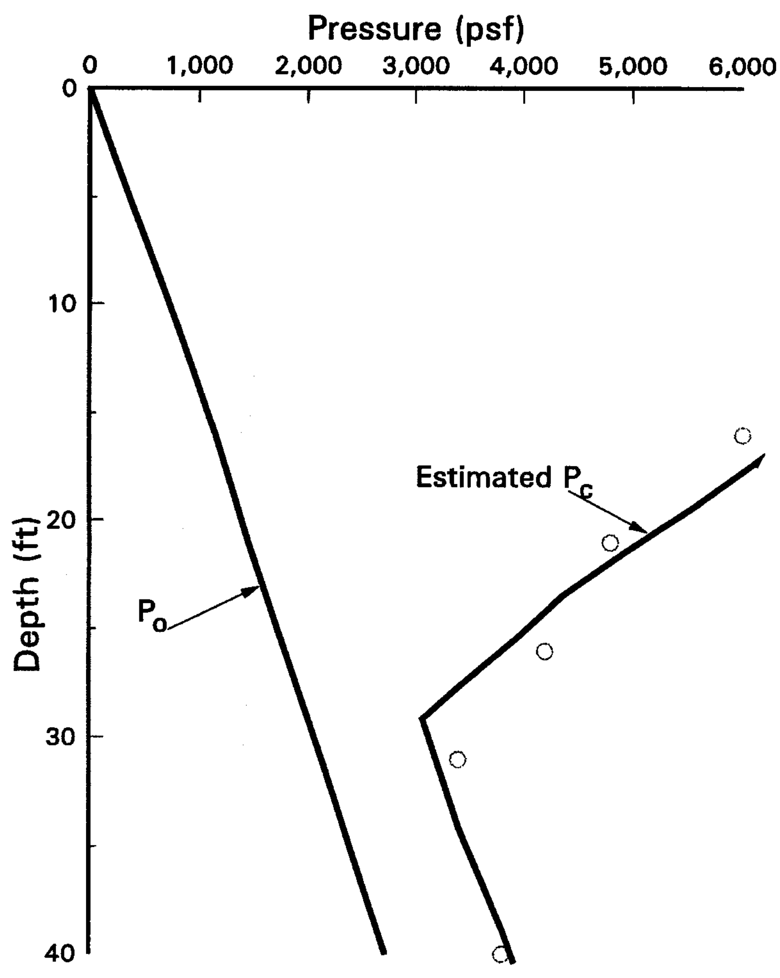
ADDITIONAL REQUESTS: *CONSOLIDATION TESTS - SPECIFIC GRAVITY ON ALL TESTS*
*STRENGTH TESTS - ATTERBURG LIMITS AND HYDROMETER ON ALL TESTS*

* CHECK MARK WHEN TEST IS PUT IN PROGRESS ✓ ; CIRCLE WHEN COMPLETED ✓ ; CROSS HATCH WHEN COMPUTED
** TO SUPPLEMENT ROUTINE CLASSIFICATION TESTS ON CONSOLIDATION AND STRENGTH TEST SAMPLES.

## CONSOLIDATION TEST RESULTS SUMMARY

**Hole UDH BAF-4**

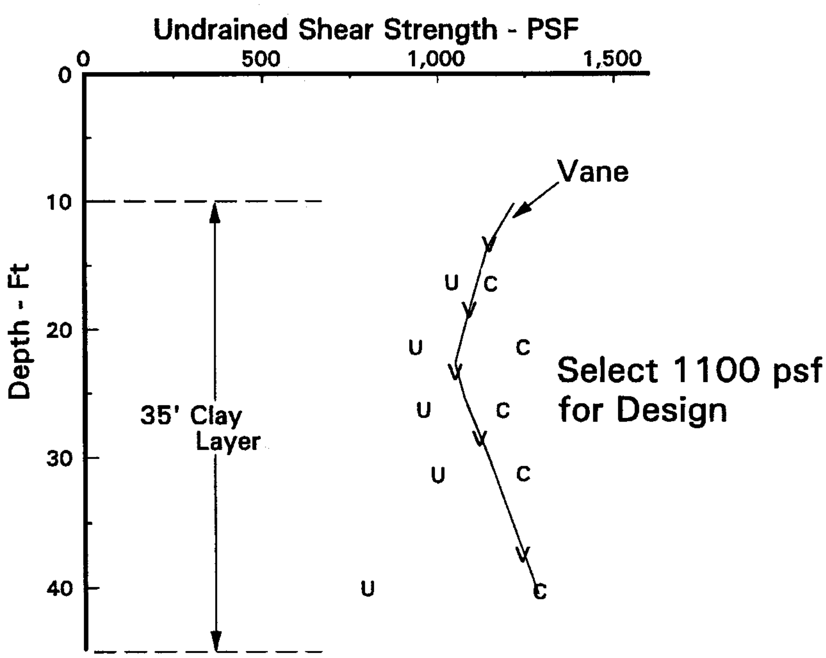
| Depth Ft. | Tube No. | w % | $P_o$, psf | $e_o$ | $P_c$, psf | $C_r$ | $C_c$ | $c_v$ |
|---|---|---|---|---|---|---|---|---|
| 11 | T3 | 33 | 800 | 0.91 | 6500 | 0.033 | 0.35 | 0.6 |
| 16 | T4 | 35 | 1150 | 0.89 | 6000 | 0.031 | 0.32 | 0.4 |
| 21 | T5 | 31 | 1450 | 0.96 | 4800 | 0.040 | 0.36 | 0.8 |
| 26 | T6 | 36 | 1790 | 1.01 | 4200 | 0.035 | 0.34 | 0.6 |
| 31 | T7 | 38 | 2130 | 0.98 | 3400 | 0.037 | 0.34 | 0.8 |
| 40 | T9 | 37 | 2720 | 1.02 | 3800 | 0.032 | 0.35 | 0.4 |



**SHEAR STRENGTH TEST RESULTS SUMMARY**

**Hole UDH BAF-4**

| | | | Undrained Strength – psf | | | |
|---|---|---|---|---|---|---|
| **Depth Ft.** | **Tube No.** | **w %** | **$U_u$ (U)** | **$C_u$ @ $P_o$ (C)** | **Vane (V)** | |
| | | | | | **Undisturbed** | **Remolded** |
| 13 | | 34 | | | 1150 | 550 |
| 16 | T4 | 34 | 1050 | 1150 | | |
| 18 | | 36 | | | 1100 | 600 |
| 21 | T5 | 35 | 950 | 1250 | | |
| 23 | | 38 | | | 1050 | 500 |
| 26 | T6 | 39 | 975 | 1200 | | |
| 28 | | 37 | | | 1125 | 550 |
| 31 | T7 | 40 | 1000 | 1250 | | |
| 37 | | 35 | | | 1250 | 600 |
| 40 | T9 | 38 | 800 | 1300 | | |



**Summary of the Laboratory Testing Phase for Apple Freeway Design Problem**

- Construct: $P_0$ Diagram

  Increase of pressure in the soil with depth.

- Prepare: Test Request

  Test pressures represent range of increase due to the embankment.

- Consolidation Results

  Compressibility, precompression and drainage rate of clay deposit.

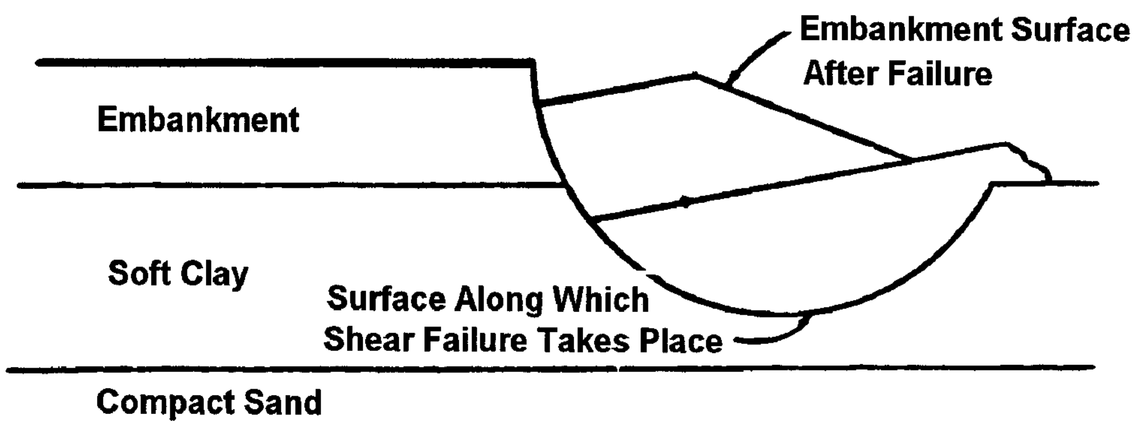
- Strength Results

  Cohesion and increase of shear strength with confining pressure found.

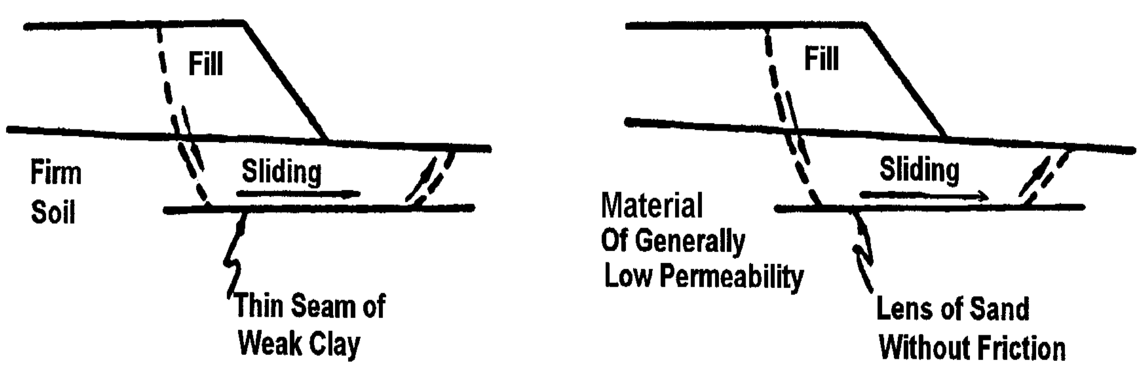
# CHAPTER 5.0
# SLOPE STABILITY

Ground stability must be assured prior to consideration of other foundation related items. Embankment foundation problems involve the support of the embankment by natural soil. Problems with embankments and structures occasionally occur which could be prevented by initial recognition of the problem and appropriate design. Stability problems most often occur where the embankment is to be built over soft weak soils such as low strength clays, silts, or peats. Once the soil profile, soil strengths, and depth of water table have been determined by both field explorations and field and lab testing, the stability of the embankment can be analyzed and factor of safety estimated.

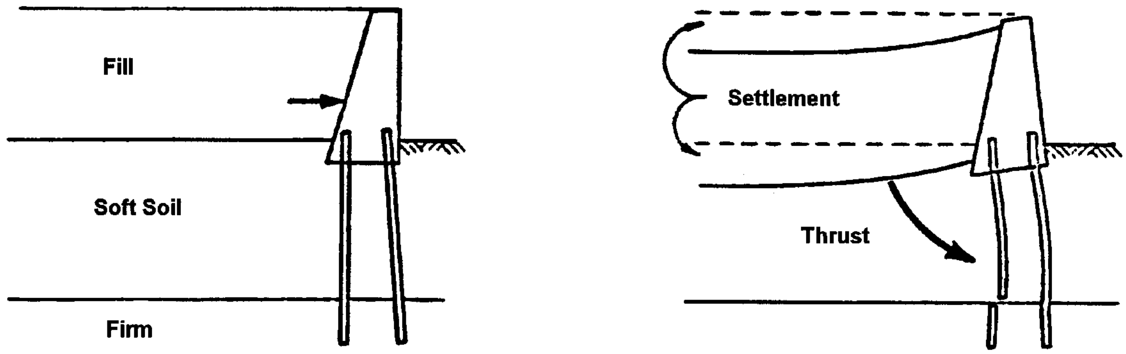
There are three major types of instability that should be considered in the design of embankments over weak foundation soils. These are illustrated in Figure 5-1.




a. Circular Arc Failure




b. Sliding Block Failure




c. Lateral Squeeze of Foundation Soil

Figure 5-1(a, b, and c): Major Types of Approach Embankment Stability Problems

Recommendations on how to recognize, analyze, and solve each of these three problems are presented in this chapter.

These stability problems as illustrated in Figure 5-1 are "external" stability problems. "Internal" embankment stability problems generally result from the selection of poor quality embankment materials and/or improper placement requirements. Internal stability may be "ordered" in project specifications by specifying granular materials with minimum gradation and compaction requirements. An example of a typical specification for approach embankment construction is shown in Chapter 6.

## 5.1 EFFECTS OF WATER ON SLOPE STABILITY

- **Importance of Water**

  Next to gravity, water is the most important factor in slope stability.

- **Effect of Water on Frictional Soils**

  In cohesionless soils, water does not affect the angle of internal friction ($\phi$). The effect of water on cohesionless soils below the water table is to decrease the intergranular (effective) pressure between soil grains which decreases the frictional shearing resistance.

- **Effect of Water on Clays**

  Routine seasonal fluctuations in the water table do not usually influence either the amount of water in the pore spaces between soil grains or the cohesion. The attractive forces between soil particles prevent water absorption unless external forces such as pile driving, disrupt the grain structure. However, certain clay minerals do react to the presence of water and cause expansion of the clay mass.

  An increase in absorbed moisture is a major factor in the decrease in strength of expansive cohesive soils (Figure 5-2). Water is absorbed by expansive clay minerals, causing high water contents which decrease the cohesion of expansive clayey soils.

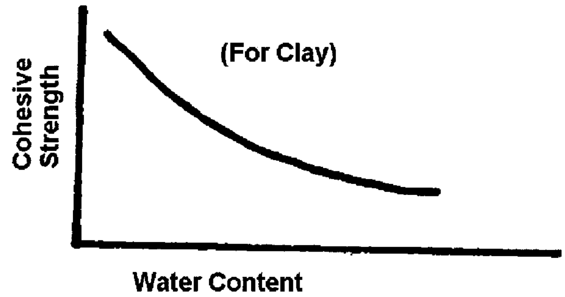



Figure 5-2: Effect of Water Content on Cohesive Strength of Clay

- **Fills on Clays**

  Excess pore pressures are created when fills are placed on clay or silt. As the pore pressure dissipates, consolidation occurs, and the clay or silt strength increases. This is the reason the factor of safety increases with time.

- **Cuts in Clay**

  As a cut is made in clay the effective stress is reduced. This will allow the clay to expand and absorb water, which will lead to a decrease in the clay strength with time. This is the reason the factor of safety of a clay cut slope decreases with time. Cut slopes in clay should be designed using effective strength parameters and the effective stress which will exist after the cut is made.

- **Slaking - Shales, Claystones, Siltstones, etc.**

  Sudden moisture increase in a dry soil can produce a pore pressure increase in trapped pore air accompanied by local soil expansion and strength decrease. The "slaking" or sudden disintegration of hard shales, claystones, and siltstones result from this mechanism. If placed as rock fill, water percolating through the fill causes these materials to disintegrate to a clay soil, which often leads to settlement and/or shear failure of the fill. Index tests such as the jar-slake test and the slake-durability test are shown in "Design and Construction of Compacted Shale Embankments," FHWA RD-78-14.

## 5.2 DESIGN FACTOR OF SAFETY

A minimum factor of safety of 1.25 is ordinarily used for highway embankment side slopes. This safety factor value should be increased to a minimum of 1.30 for slopes whose failure would cause significant damage such as end slopes beneath bridge abutments, major retaining structures, etc. The selection of the actual safety factor to be used on a particular project depends on:

- Stability analysis method used.
- Method of shear strength determination.
- Confidence in reliability of subsurface data.
- Consequences of failure.

## 5.3 CIRCULAR ARC FAILURE

Experience and observations of failures of embankments built over relatively deep deposits of soft foundation soils have shown that when failure occurs, the embankment sinks down, the adjacent ground rises and the failure surface follows a circular arc as illustrated in Figure 5-3.

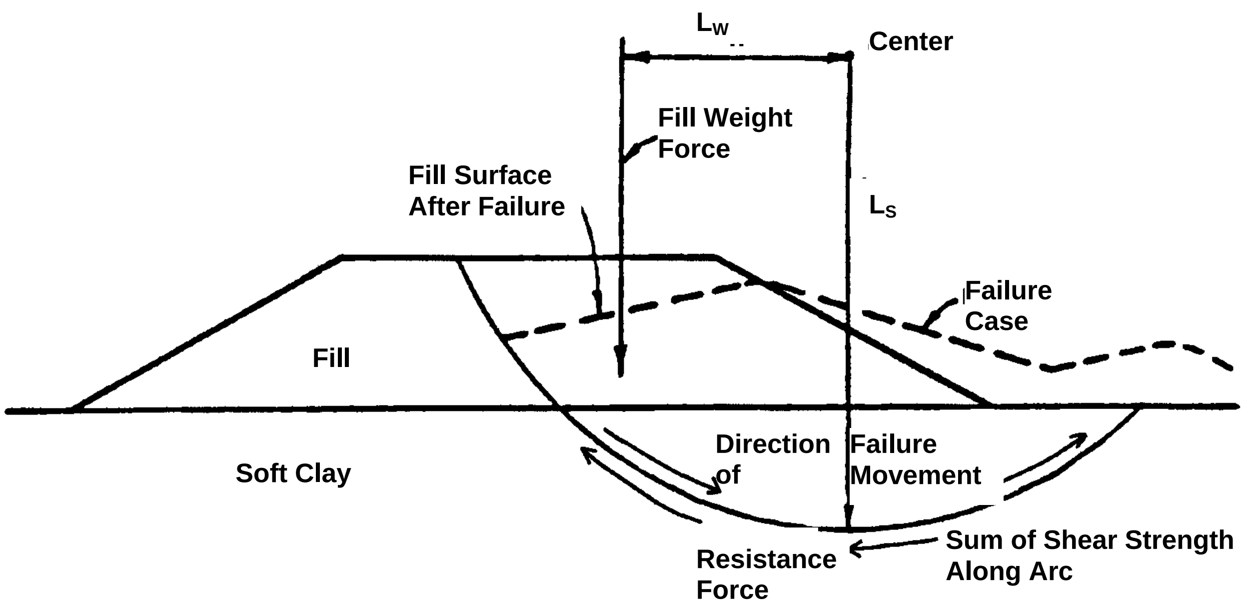



Figure 5-3: Typical Circular Arc Failure Mechanism

The failure force (driving force) consists of the weight of the embankment. The overturning moment is the product of the weight of the embankment (acting through its center of gravity) times the lever arm distance to the center of rotation ($L_W$).

The resisting force against movement is the sum of all soil shear strength (friction and cohesion) acting along the failure arc. The resisting moment is the product of the shear strength times the radius of the circle ($L_S$).

The factor of safety against overturning is equal to the ratio of the resisting moment to overturning moment.

$$\text{Factor of Safety} = \frac{\text{Total Shear Strength} \times L_S}{\text{Weight Force} \times L_W} = \frac{\text{Resisting Moment}}{\text{Overturning Moment}} \quad (5\text{-}1)$$

When the factor of safety is less than 1, failure will take place.

### 5.3.1 Simple Rule of Thumb for Factor of Safety

A simple rule of thumb based on simplified bearing capacity theory can be used to make a preliminary "guestimate" of the factor of safety against circular arc failure for an embankment built on a clay foundation.

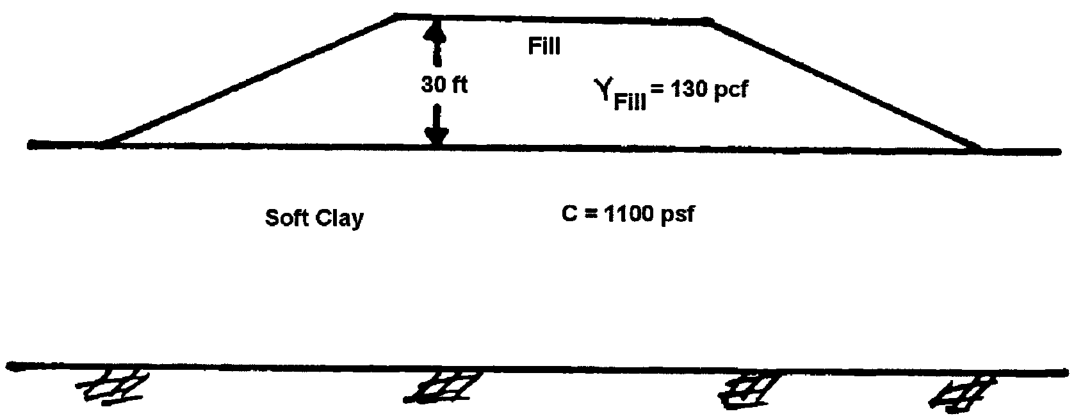
The rule of thumb is:

$$\text{Factor of Safety (F.S.)} \cong \frac{6C}{\gamma_{Fill} \times H_{Fill}} \tag{5-2}$$

Where: $C$ = Cohesion Strength of Foundation Clay (psf)
$\gamma_{Fill}$ = Fill Soil Unit Weight (pcf)
$H_{Fill}$ = Fill Height (Feet)

For example, consider the following proposed embankment.

$$\text{F.S.} = \frac{(6)(1100 \text{ psf})}{(130 \text{ pcf})(30')} = 1.69 \quad \text{Using Rule of Thumb (Equation 5-2)}$$




**The factor of safety computed using this rule of thumb should never be used for final design**. The simple equation obviously does not take into account such factors as fill strength or fill slope angle and does not identify the location of a critical failure surface. If the factor of safety using the rule of thumb is less than 2.5, a more sophisticated stability analysis is required.

However, this rule of thumb can be helpful very early in the design stage to make a quick preliminary check on whether stability may be a problem and if more detailed analyses should be conducted. It can also be of use in the field while the boring and sampling is being done. For example, if in situ vane shear tests are being carried out as part of the field investigation for a proposed embankment, the vane strength can be used with the rule of thumb equation, by the soils engineer or geologist, to estimate the F.S. right in the field. This can aid in directing the drilling, sampling, and testing program while the drill crew is at the site and help insure that critical strata are adequately explored and sampled. Finally, the simple rule of thumb factor of safety can be used to check for gross errors in computer output or input.

### 5.3.2 Stability Analysis Methods (General)

There are several available methods that can be used to perform a circular arc stability analysis for an approach embankment over soft ground. The simplest most basic method is known as the NORMAL METHOD OF SLICES. The normal method of slices can easily be performed by a hand solution and is also a method by which the computation of driving and resisting forces is straightforward and easily demonstrated. For this method, the failure surface is assumed to be the arc of a circle as shown in Figure 5-4 and the factor of safety against sliding along the failure surface is defined as the ratio of the moment of the available soil shear strength resisting forces (friction plus cohesion) on the trial failure surface to the net moment of the driving forces (due to the embankment weight), that is:

$$\text{F.S.} = \frac{\text{Sum of Resisting Forces} \times \text{Moment Arm (R)}}{\text{Sum of Driving Forces} \times \text{Moment Arm (R)}} \qquad (5\text{-}3)$$

Note that since the method consists of computing the driving and resisting forces along (parallel) to the failure arc, the moment arm R is the same for both the driving and resisting forces, thus, R cancels out of the factor of safety equation and the equation reduces to:

$$\text{F.S.} = \frac{\text{Sum of Resisting Forces}}{\text{Sum of Driving Forces}} \qquad (5\text{-}3a)$$

The free body diagram (Figure 5-4) shows the failure surface is divided into slices and the following basic assumptions are made:

1. The available shear strength of the soil can be adequately described by the Mohr-Coulomb equation:

$$S = C + (\sigma - \mu) \tan \phi$$

Where: S = Total shear strength
C = Cohesion component of shear strength
$(\sigma - \mu) \tan \phi$ = Frictional component of shear strength
$\sigma$ = The total normal stress against the failure surface slice base due to the weight of soil and water above the failure surface
$\mu$ = Water uplift pressure against the failure surface
$\phi$ = Soil angle of internal friction
Tan $\phi$ = Coefficient of friction along failure surface

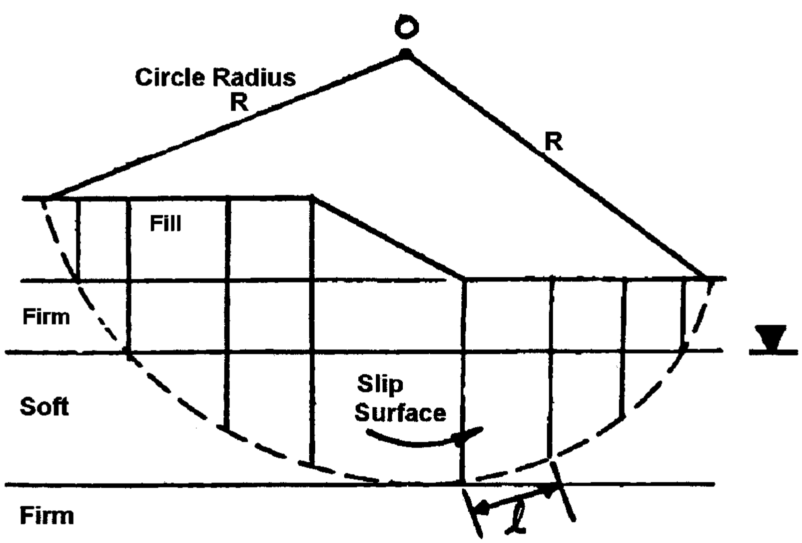



Figure 5-4: Geometry of Normal Method of Slices

2. The factor of safety is the same for all slices.

3. The factors of safety with respect to cohesion (C) and friction (tan $\phi$) are equal.

4. All forces (shear and normal) on the sides of each slice are ignored.

5. The water pressure ($\mu$) is taken into account by reducing the total weight of the slice by the water uplift force acting against the slice base.

Lastly, the convention to be used in the stability analysis should be chosen. In soil problems involving water, the engineer may compute the normal and tangential forces using either total soil weights and boundary water forces (both buoyancy and unbalanced hydrostatic forces) or submerged (buoyant) soil weights and unbalanced hydrostatic forces. The results are the same. When total weight and boundary water forces are used, the equilibrium of the entire block is considered. When submerged weights and hydrostatic forces are used, the equilibrium of the mineral skeleton is considered. The total weight notation is used herein as this method is the simplest to compute.

### 5.3.3 Normal Method of Slices; Step-By-Step Computation Procedure

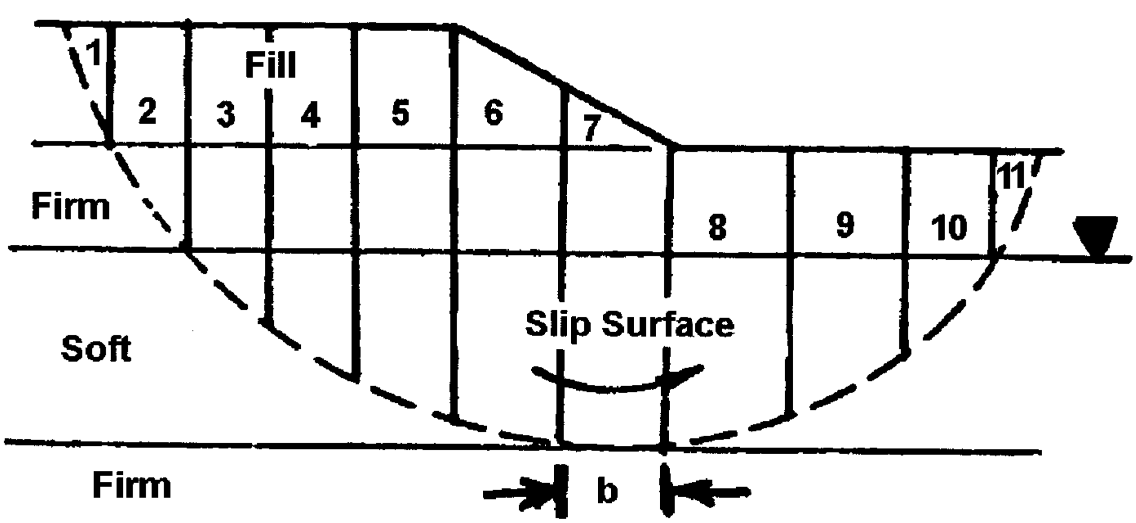
To compute the factor of safety for an embankment using the normal method of slices, the step-by-step computational procedure is as follows:

(Note: An example of the method of slices hand solution is shown for the Apple Freeway Design Example – Slope Stability)

**Step 1. Draw cross-section of embankment and foundation soil profile using either 1" = 10 feet or 1" = 20 feet scale both horizontal and vertical.**

**Step 2. Select a circular failure surface such as shown in Figure 5-4.**

**Step 3. Divide the circular mass above the failure surface into 10 - 15 vertical slices as illustrated below:**



To simplify computation, locate the vertical sides of the slices so that the bottom of any one slice is located entirely in a single soil layer or at the water level - circle intersection, and locate vertical slice top boundaries at breaks in the slope. The slice widths do not have to be equal. For convenience assume a one-foot thick section of embankment (this simplifies computation of driving and resisting forces).

Also as shown in Figure 5-5 and 5-6 the driving and resisting forces of each slice act at the intersection of a vertical line drawn from the center of gravity of the slice to establish a centroid point on the circle. Lines (called rays) are then drawn from the circle center to intersect the circle at the centroid point. The α angles are then measured from the vertical to each ray.

When the water table is sloping, use equation 5-4 to calculate the water pressure on slice base:

$$\mu = h_w \gamma_w \cos^2 \alpha_w \qquad (5\text{-}4)$$

Where: $\alpha_w$ =slope of water table from horizontal in degrees

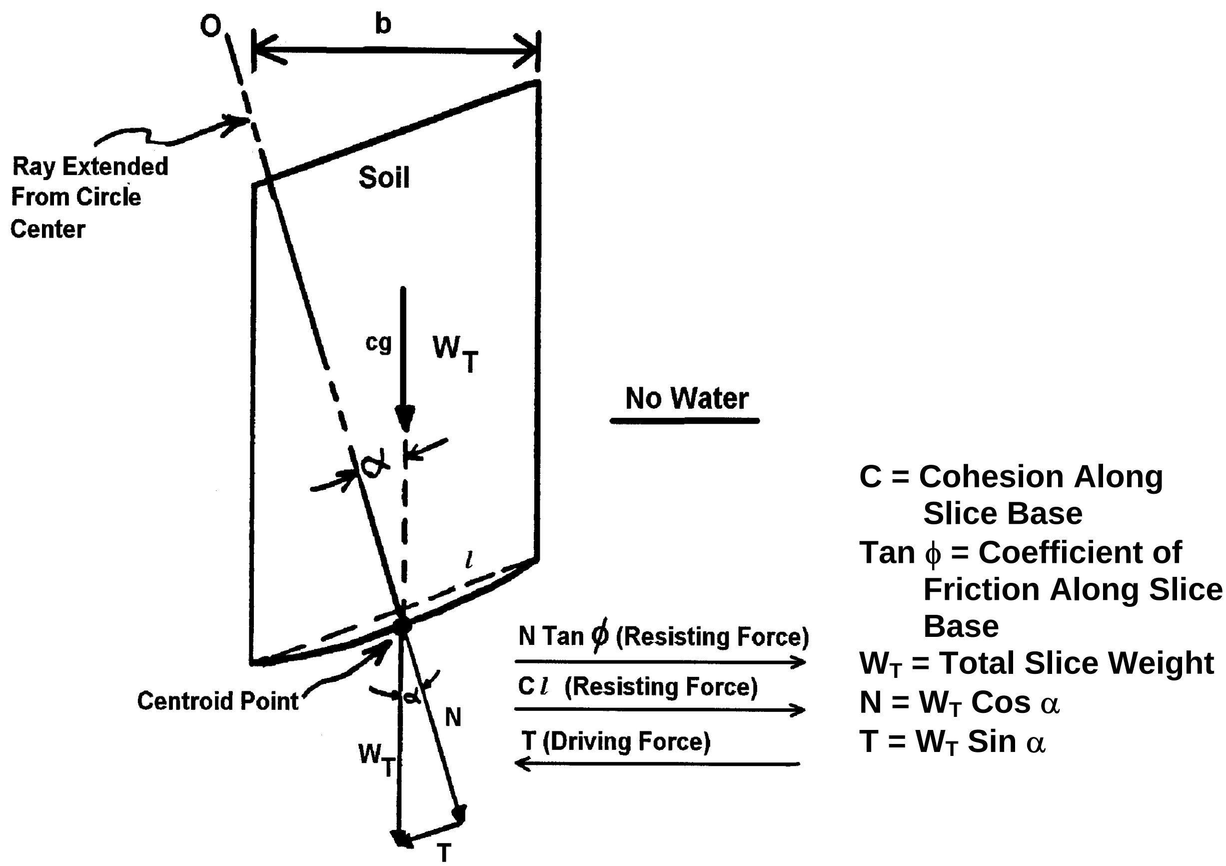



Figure 5-5: Forces on A Slice without Water Effect

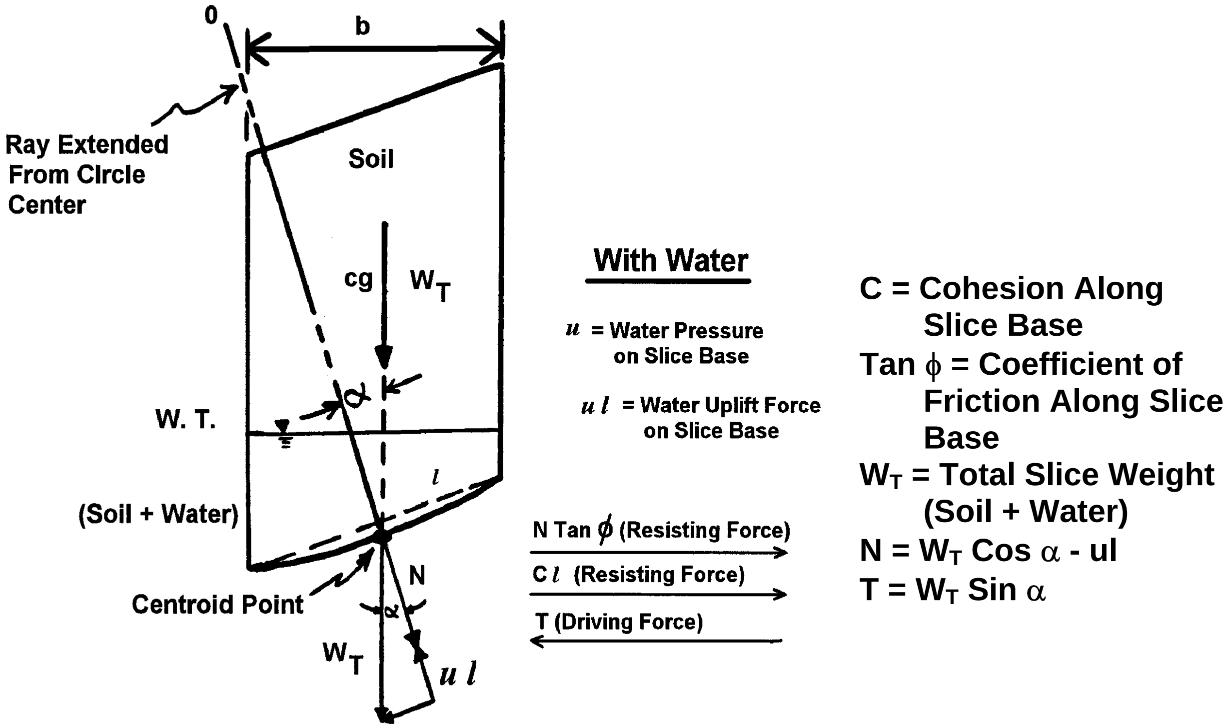



Figure 5-6: Forces on A Slice with Water

**Step 4: Compute the total weight ($W_T$) of each slice.**

For illustration, the resisting and driving forces acting on individual slices with and without water pressure are shown on Figures 5-5 and 5-6.

To compute $W_T$, use total soil unit weight ($\gamma_{\text{Total}}$) both above and below the water table.

$$W_T = \gamma_{\text{Total}} \times \text{Average Slice Height} \times \text{Slice Width (b)} \qquad (5\text{-}5)$$

For example: Assuming $\gamma_{\text{Total}} = 120$ pcf
Average Slice Height = 10 ft
Slice Width = 10 ft

Then $W_T = (120)\ (10)\ (10) = 12{,}000$ lbs.

**Step 5: Compute N Tan ϕ (Frictional resisting force) for each slice.**

$$N = W_T \text{ Cos } (\alpha) - ul \qquad (5\text{-}6)$$

$N$ = Effective normal force against the slice base (force between granular soil grains)
$W_T$ = Total slice weight (from 4 above)
$\alpha$ = Angle between vertical and line drawn from circle center to midpoint of slice base (note it is also equal to angle between the horizontal and a line tangent to the slice base)
$\mu$ = Water pressure on slice base (average height of water, $h_w \times \gamma_{\text{water}}$)

$l$ = Arc length of slice base

To simplify computations, take $l$ as the straight-line distance along the slice base and use $\gamma_{water}$ = 60 pcf.

$\mu\, l$ = Water uplift force against slice base
$\phi$ = Soil friction angle
Tan $\phi$ = Coefficient of friction along slice base

Note that the effect of water is to reduce the normal force against the slice base and thus reduce the frictional resisting force ($N \tan \phi$). To illustrate this, take the same slice used in step 4 and compute $N \tan \phi$ for the slice with no water and then for the water table located 5 feet above the slice base.

Assume: $\phi = 25^o$
$\alpha = 20^o$
$l = 11$ ft

Example: If using Equation 5-6 with no water in slice:

$\mu\, l = 0$
$N = W_T \cos \alpha = (12{,}000 \text{ lbs.})(\cos 20^o) = 11{,}276$ lbs.
$N \tan \phi = (11{,}276 \text{ lbs}) (\tan 25^o) = \underline{5{,}258 \text{ lbs}}$.

If with water 5 ft. above slice base:

$\mu\, l = (h_w)(\gamma_w)(l) = (5)(60)(11) = 3{,}300$ lbs.
$N = W_T \cos \alpha - \mu\, l = 11{,}276 - 3{,}300 = 7{,}976$ lbs.
$N \tan \phi = (7{,}976)(\tan 25°) = \underline{3{,}719 \text{ lbs.}}$

**Step 6: Compute Cl (resisting force due to cohesion for each slice).**

$C$ = cohesive soil strength
$l$ = length of slice base

Example: $C$ = 200 psf
$l$ = 11 ft
$Cl = (200)(11) = 2{,}200$ lbs.

**Step 7: Compute T (tangential driving force).**

$$T = W_T \text{ Sin } \alpha \quad (5\text{-}7)$$

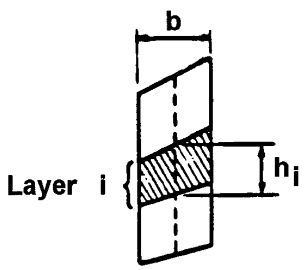
T is the component of total slice weight ($W_T$) acting tangent to the slice base. T is the driving force due to the weight of both soil and water in the slice.

Example: Given $W_T$ = 12,000 lbs.
$\alpha = 20^o$
$T = W_T \sin \alpha = (12{,}000 \text{ lbs.})(\sin 20^o) = 4{,}104$ lbs.

**Step 8: Sum resisting forces and driving forces for all slices and compute factor of safety.**

$$F.S. = \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}} = \frac{\sum N \,\text{Tan}\phi + \sum C\,1}{\sum T} \qquad (5\text{-}3a)$$

Tabular computation forms for use in performing a method of slices stability analysis by hand are included on Figures 5-7 and 5-8.




$\gamma_i$ = unit weight of layer i
$h_i$ = height of layer at center of slice
$W_i$ = partial weight = b $h_i$ $\gamma_i$
$\sum W_i$ = total weight of slice $W_T$

| Slice No. | b | $h_I$ | $\gamma_I$ | $W_i$ | $\sum W_i = W_T$ |
|---|---|---|---|---|---|
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Figure 5-7: Tabular Form for Computing Weights of Slices

| Slice No. | $W_T$ | $\ell$ | $\alpha$ | C | $\phi$ | u | $u\ell$ | $W_T \cos\alpha$ | $N = W_T \cos\alpha - u\ell$ | $N \tan = (W_T \cos\alpha - u\ell)\tan\phi$ | $c\ell$ | $T = W_T \sin\alpha$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
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| | | | | | | | | | $\Sigma$ | | | |


C = cohesion intercept
$\phi$ = friction angle
u = pore pressure

} at base of slice

$W_T$ = Total Weight of Slice (Soil+Water)

$$F = \frac{\Sigma (W_T \cos\alpha - \mu\ell) \tan\phi + \Sigma c\ell}{\Sigma W_T \sin\alpha} = \frac{\Sigma N \tan\phi + \Sigma c\ell}{\Sigma T}$$

F = ______________ =

Figure 5-8: Tabular Form for Calculating Factor of Safety by Normal Method of Slices

### 5.3.4 Recommended Stability Methods

There are many other stability analysis methods available besides the NORMAL method - such as Bishop method, Janbu, etc. These methods are primarily variations and refinements of the basic method of slices. The differences in the more refined methods lie in the assumption made regarding the shear and normal forces made on the sides of slices. For example, the NORMAL method assumes the vertical and horizontal slice side forces are zero. The Bishop method, by comparison, includes the horizontal slice side force and ignores the vertical slice side force. For purely cohesive clay soils the NORMAL and Bishop methods will give identical results. For soils which have frictional strength, the Bishop method should be used. The NORMAL method is more conservative and will give unrealistically lower factors of safety than the Bishop or other more refined methods. While none of the methods are 100 percent theoretically correct, currently available procedures are sufficiently accurate for practical analysis and design.

The method of analysis, which should be used to determine a factor of safety, depends on the soil type, the source of and confidence in the soil strength parameters, and the type of slope that is being designed. Soil design analyses should only be performed by qualified experienced geotechnical personnel. Design criteria recommended for analysis of Slope Stability are given in Table 5-1.

**TABLE 5 -1**
**SLOPE STABILITY DESIGN CRITERIA**

| Foundation Soil Type | Type of Analysis | Source of Strength Parameters | Remarks |
|---|---|---|---|
| Cohesive | Short-term (embankments on soft clays – immediate end of construction). | UU or field vane shear test or CU triaxial test, (undrained strength parameters at $P_o$. $\phi = 0$ analysis). | Use Bishop method. An angle of internal friction should not be used to represent an increase of shear strength with depth. The clay profile should be broken into convenient layers and the appropriate cohesive shear strength assigned to each layer. |
| Cohesive | Stage construction (embankments on soft clays – build embankment in stages with waiting periods to take advantage of clay strength gain due to consolidation. | CU triaxial test. Some samples have to be consolidated to higher than existing in situ stress to determine clay strength gain due to consolidation under staged fill heights. (Undrained strength parameters at appropriate $P_o$ for staged height | Use Bishop method at each stage of embankment height. Consider that clay shear strength will increase with consolidation under each stage. Consolidation test data needed to estimate length of waiting periods between embankment stages. Instrumentation (piezometers and settlement devices) should be used to monitor pore pressure dissipation and consolidation during construction. |
| Cohesive | Long-term (embankment on soft clays and clay cut slopes). | CU triaxial test with pore pressure measurements or CD triaxial test (effective strength parameters). | Use Bishop analysis with combination of cohesion and angle of internal friction (effective strength parameters from laboratory test). |
| Cohesive | Existing failure planes. | Direct shear or direct simple shear test. Slow strain rate and large deflection needed. Residual strength parameters. | Use Bishop, Janbu or Spencer's method to duplicate previous shear surface. |
| Granular | All types. | Get effective friction angle from charts of standard penetration resistance (SPT) versus friction angle or from direct shear tests. | Use Bishop Method with an effective stress analysis. |

*UU= unconsolidated undrained; CU= consolidated undrained;
CD= consolidated drained;
$P_o$ = in situ vertical effective overburden pressure

### 5.3.5 Stability Charts

Slope stability charts are available which are sometimes useful for preliminary analysis; such as to compare alternates which can later be examined by more detailed analyses. One of the major shortcomings is that most stability charts are for ideal, homogeneous soil conditions which are not encountered that often in practice.

The interested reader is referred to the Navy Design Manual (NAVFAC DM-7.1) or Terzaghi and Peck (1967) for examples of stability charts and their use.

### 5.3.6 Remarks on Safety Factor

For normal highway embankment side slopes, a minimum design safety factor of 1.25 is ordinarily used. For slopes which would cause greater damage upon failure, such as end slopes beneath bridge abutments, major retaining structures, etc., the design safety factor should be increased to at least 1.30. For cut slopes in fine-grained soils which can lose shear strength with time, a safety factor of 1.5 is desirable.

## 5.4 CRITICAL FAILURE SURFACE

The step-by-step procedure presented on the preceding pages shows how to compute the factor of safety for one selected circular arc failure surface. The complete analysis requires that a large number of assumed failure surfaces be checked in order to find the most critical one; the surface with the lowest factor of safety. This would obviously be a tedious and time consuming operation if done by hand.

This is where the computer becomes such a valuable design tool. The stability analysis is easily adapted to computer solution. A grid of possible circle centers is defined, and a range of radius values established for each. The computer can be directed to print out all the safety factors or just the minimum one (and its radius) for each circle center. A plot of minimum safety factor for each circle center in the form of contours can be used to define the location of the most critical circle and the minimum safety factor as shown in Figure 5-9.

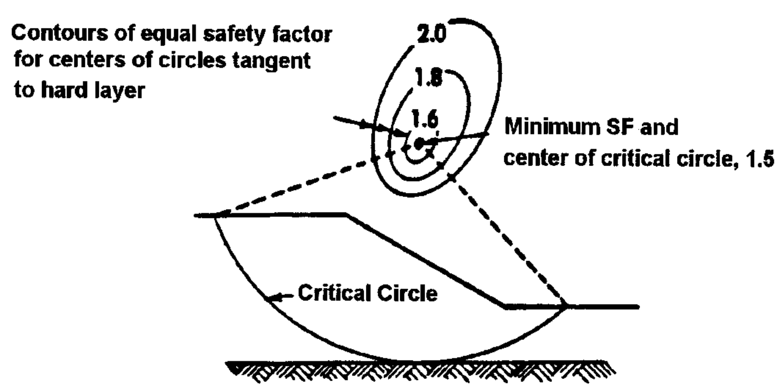



Figure 5-9: Location of Critical Circle by Plotting Contours of Minimum Safety Factors for Various Trial Circles

## 5.5 SLOPE STABILITY ANALYSIS - COMPUTER PROGRAMS

Slope stability procedures are well suited to computer analysis due to the interactive nature of the solution. Also, the simplified hand solution procedures do not properly account for interslice forces, irregular failure surfaces, seismic forces, and external loads such as line load surcharges or tieback forces. Several user-friendly micro-computer programs now exist to accurately analyze two dimensional slope stability problems. More complex computer programs are available for three dimensional slope stability analysis.

Highway agencies should, as a minimum, use a basic two-dimensional slope stability program. Desirable geotechnical features of such a program should include:

- Multiple analysis capability
  a. Circular arc (Modified Bishop)
  b. Non-circular (Janbu)
  c. Sliding block

- Variable Input Parameters
  a. Heterogeneous soil systems
  b. Pseudo-static seismic loads
  c. Tieback forces
  d. Piezometric levels

- Random generation of multiple failure surfaces with option to analyze a specific failure surface.

Desirable software features include:

- User-friendly input screens including a summary screen showing the cross section and soil boundaries in profile.

- Help screens and error tracking messages.

- Expanded output option of both resisting forces in friction, cohesion or tieback computations and driving forces in static or dynamic computations.

- Ordered output and plot of 5 minimum failure surface safety factors.

- Documentation of program.

A major problem for software users is technical support, maintenance and update of programs. Slope stability programs are in a continual process of improvement which can be expected to continue indefinitely. Highway agencies should only implement software which is documented and which the seller agrees to provide full technical support, maintenance and update. The web page for the FHWA Geotechnical Group, *www.fhwa.dot.gov/bridge/geo.htm*, contains links to distributors of FHWA software.

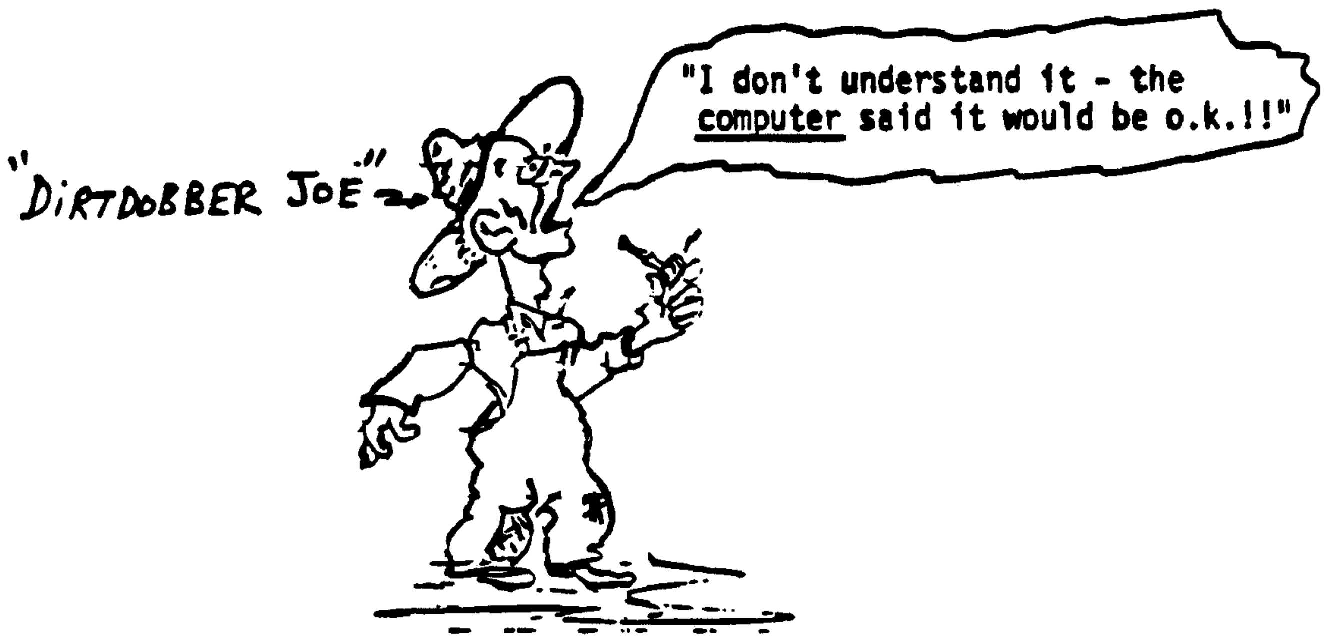
Other private firms exist which provide similar services for slope stability programs such as the STABL series, XSTABL, the UTEXAS series, etc.

IMPORTANT! IMPORTANT! IMPORTANT!

In DESIGN - Put the major emphasis where it belongs, which is on:

- Investigation
- Sampling
- Testing
- Development of Soil Profile
- Design Soil Strengths
- Water Table Location

Computer programs are only tools which aid us in the design - the answers are only as good as the input data. Don't get carried away with plugging the numbers. You may learn the "garbage in - garbage out" principle the hard way - like "Dirtdobber Joe"!




### 5.6 SLIDING BLOCK FAILURE

A "sliding block" type failure can occur (1) where the foundation soil contains thin seams of weak clay or organic soils, (2) where a shallow layer of weak soil exists at the ground surface and is underlain by firm soil, and (3) where the foundation soil contains thin sand or silt lenses sandwiched between more impermeable soil. The weak layer or lense provides a plane of weakness along which sliding can occur. In the case of sand or silt lenses trapped between impervious soil, the mechanism that can cause sliding is as follows: As the fill load is placed, the water pressure is increased in the sand or silt lense. Since the water cannot escape due to the impermeable soil above and below, the sand or silt loses frictional strength as a result of the intergranular effective stress between soil grains being decreased due to the water pressure. These problems are illustrated in Figure 5-15.

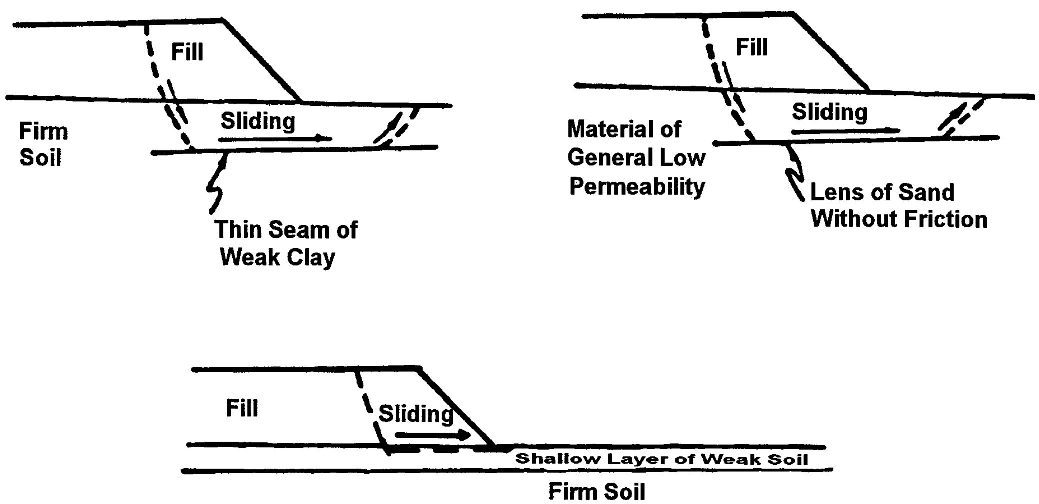



Figure 5-15: Sliding Block Failure Mechanism

When sliding occurs, an active wedge type failure occurs through the fill (similar to the active wedge that forms behind a retaining wall), and a passive wedge type failure occurs below the fill toe as soil in the toe area is pushed up out of the way. The sliding mass moves essentially as a block, thus the term "sliding block."

## 5.7 SLIDING BLOCK – HAND METHOD OF ANALYSIS

A simple sliding block analysis to estimate factor of safety against sliding is straightforward and can be easily and quickly performed by hand. For the analysis, the potential sliding block is divided into three parts; (1) An active wedge at the head of the slide, (2) A central block, and (3) A passive wedge at the toe. For example see figure 5-16.

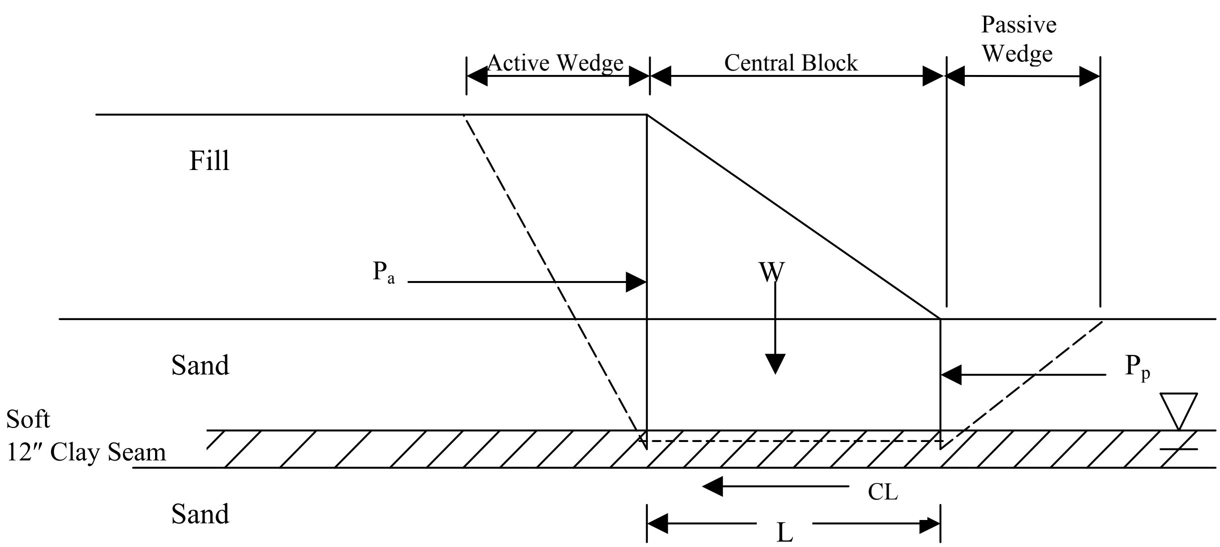



Figure 5-16: Geometry and Parameters for Sliding Block Mechanism

For the problem illustrated in Figure 5-16 above, the factor of safety would be computed by summing forces horizontally, to give:

$$F.S. = \frac{\text{Horizontal Resisting Forces}}{\text{Horizontal Driving Forces}} = \frac{P_P + CL}{P_a} \tag{5-8}$$

Where:
- $P_a$ = Active Force (Driving)
- $P_p$ = Passive Force (Resisting)
- CL = Resisting Force due to cohesion of clay

(For convenience of computation of 1 foot thick slice of embankment is assumed.)

Several trial locations of the active and passive wedges must be checked to determine the minimum factor of safety. Note that since wedge type failures occur at the head and toe of the slide, similar to what occurs behind retaining walls, the active and passive forces are taken as acting against vertical planes which are treated as "imaginary" retaining walls, and the active and passive forces are computed the same as for retaining wall problems.

Computation of Forces - Simple Sliding Block Analysis:

For the simple sliding block type problem illustrated on the previous page the forces used in the factor of safety computation can be calculated as follows using the Rankine approach:

Driving Force

$$P_a = 1/2\ \gamma\ H^2\ K_a \tag{5-9}$$

Where:
- $P_a$ = Active force (kips)
- $\gamma$ = Soil unit weight (kcf)
- H = Height of soil layer in active wedge (ft)
- $K_a$ = Active earth pressure coefficient for level ground surface
  $K_a = \tan^2(45^o - \phi/2)$
- $\phi$ = Soil angle of internal friction

Resisting Force

$$P_p = 1/2\ \gamma\ H^2\ K_p \tag{5-10}$$

Where:
- $P_p$ = Passive Force (kips)
- $\gamma$ = Soil Unit Weight (kcf)
- H = Height of soil layer in passive wedge (ft)
- $K_p$ = Passive earth pressure coefficient for level ground surface
  $K_p = \tan^2(45^o + \phi/2)$

Resisting Force (CL in kips) = Clay cohesion (C in ksf) X Length of central wedge (L in feet)

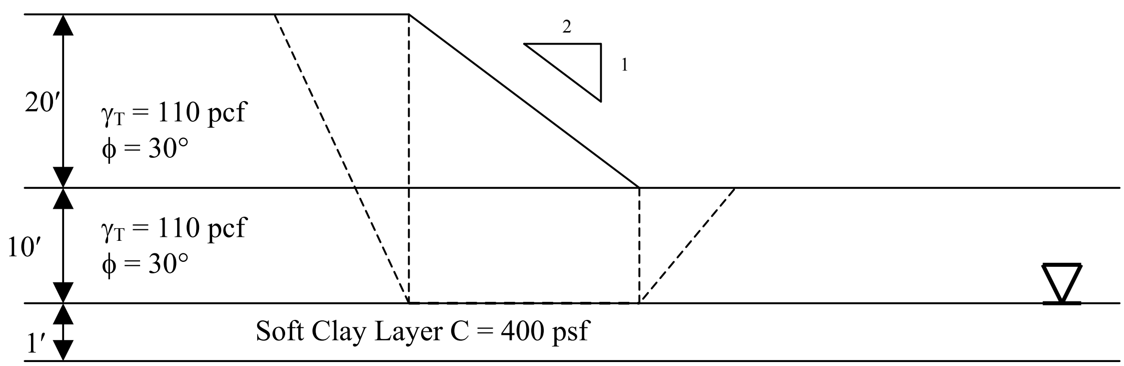
Computation Tips:

These are two important design tips that should be kept in mind when performing a sliding block analysis.

First, be aware that if the active or passive wedge passes through more than one soil type with different soil strengths or soil weights, then the active or passive pressure changes as you go from one soil layer into the next (due to change in either the soil weight and/or the earth pressure coefficient $K_a$ or $K_p$). The easiest way to handle this is to first compute the active or passive pressure diagram, then compute the active or passive force from the area of the pressure diagram.

Second, when computing the active or passive pressure, remember to use buoyant (effective) soil unit weight below the water table.

**Example 5.1:** Find the Safety Factor For The 20′ High Embankment By The Simple Sliding Block Method Using Rankine Pressure Coefficients, for the Slope Shown Below.




Firm Material

**Solution:**

**Step 1: Compute Driving Force ($P_a$)**

- Active Driving Force ($P_a$) (consider a 1 ft. wide strip of the embankment)

$$P_a = \frac{1}{2}\gamma_T H^2 K_a \qquad \text{(use } \gamma_T \text{ as the water table is below the failure plane)}$$

$$K_a = \text{Tan}^2(45 - \frac{\phi}{2}) = \text{Tan}^2(45 - \frac{30}{2}) = 0.33$$

$$P_a = \frac{1}{2}(0.110\,kcf)(30')^2(0.33)(1') = 16.5^K$$

**Step 2: Compute Resisting Force (Cl & $P_p$)**

- Central Block Resistance (Cl)

$$Cl = (0.400\,kcf)(40')(1') = 16.0^K$$

- Passive Resisting Force ($P_p$)

$$P_p = \frac{1}{2}\gamma_T H^2 K_p$$

$$K_p = \mathrm{Tan}^2(45 + \frac{\phi}{2}) = \mathrm{Tan}^2(45 + \frac{30}{2}) = 3.0$$

$$P_p = (\frac{1}{2})(0.110\mathrm{kcf})(10)^2(3.0)(1') = 16.5^K$$

$$\text{Safety Factor} = \frac{Cl + P_p}{P_a} = \frac{16.0^K + 16.5^K}{16.5^K} = 1.97$$

## 5.8 COMPUTATION OF FORCES - COMPLICATED SLIDING BLOCK ANALYSIS

The Rankine approach is a useful tool to portray the mechanism of a planar failure condition. However a general force diagram applicable to a more difficult sliding block type problem can account for the effects of water pressure, cohesion, friction, and a sloping failure plane in the analysis. This analysis procedure, which is described in FHWA-SA-94-005, can be used both to estimate factor of safety for assumed failure surfaces in design or to "backanalyze" sliding block type landslide problems.

Computer solutions are also available for defined planar surface or non-circular surface failure modes. However most of those solutions do not use the simplified Rankine block approach but a more complex Janbu approach to the planar failure. In general a computer solution is preferred for these planar failure problems.

## 5.9 DESIGN SOLUTIONS - STABILITY OF EMBANKMENTS

There are usually several solutions to a stability problem. The one chosen should be the most economical considering the following factors:

1. Available materials.
2. Quantity and cost of materials.
3. Construction time schedules.
4. Line and grade requirements.
5. Right-of-way.

### 5.9.1 Embankment Stability Design Solutions

**TABLE 5-2**
**PRACTICAL DESIGN SOLUTIONS TO EMBANKMENT STABILITY PROBLEMS**

| | |
|---|---|
| *1. Relocate highway alignment. | A line shift of the highway to a better soils area may be the most economical solution. |
| *2. Reduce grade line. | A reduction in grade line will decrease the weight of the embankment and may provide stability. (Figure 5-10) |
| 3. Counterweight berms. | The weight of a counterweight berm as illustrated in (Figure 5-11), being on the outside of the center of rotation, provides an increased moment which resists failure. This increases the factor of safety. Berms should be built concurrently with the embankment. The embankment should never be completed prior to berm construction, since the critical time for shear failure is at the end of embankment work. The top surface of a berm should be sloped to drain water away from the embankment. Also care should be exercised in selection of materials and compaction requirements to assure the design unit weight will be achieved for berm construction. |
| 4. Excavation of soft soil and replacement with shear key. | The strength of soft soil is often insufficient to support embankments. In such cases, soft soils are excavated and replaced with granular material (Figure 5-12). |
| 5. Displacement of soft soil. | For deep soft deposits, excavation is difficult. The soft soil can be displaced by generating continuous shear failures along the advancing fill front until the embankment is on firm bottom. The mudwave forced up in front of the fill must be excavated to insure continuous displacement and prevent large pockets of soft soil from being trapped under the fill. |
| 6. Slow rate or stage construction. | Many weak subsoils will tend to gain strength during the loading process as consolidation occurs and pore water pressures dissipate. For soils that consolidate relatively fast, such as some silts and silty clays, this method is practical. Proper instrumentation is desirable to monitor the state of stress in the soil during the loading period to insure that loading does not proceed so rapidly as to cause a shear failure. Typical instrumentation consists of slope inclinometers to monitor stability, piezometers to measure porewater pressure, and settlement devices to measure amount and rate of settlement. Planning of the instrumentation program and data interpretation should be done by a qualified geotechnical engineer. |
| 7. Lightweight embankment. | In some areas of the country, lightweight blast furnace slag, shredded rubber tires, expanded polystyrene blocks, or expanded shale is available. The slag material weighs about 80 pcf. Sawdust fill weighs about 50 pcf and has friction angle of $35^{o}$ or more. Shredded tires and EPS are even lighter materials. The overturning force is decreased by the lighter embankment weight. Typical Specifications for lightweight fills used by the NYDOT and WashDOT are included in Appendix C and D. |
| 8. Ground improvement | The use of recently developed techniques such as stone columns, soil mixing, geosynthetics, soil nailing, ground anchors, and grouting can be used to increase resisting forces. Specialty contractors should be considered for these design solutions. |

*Always considers these simple solutions first to avoid more complicated, expensive solutions which follow

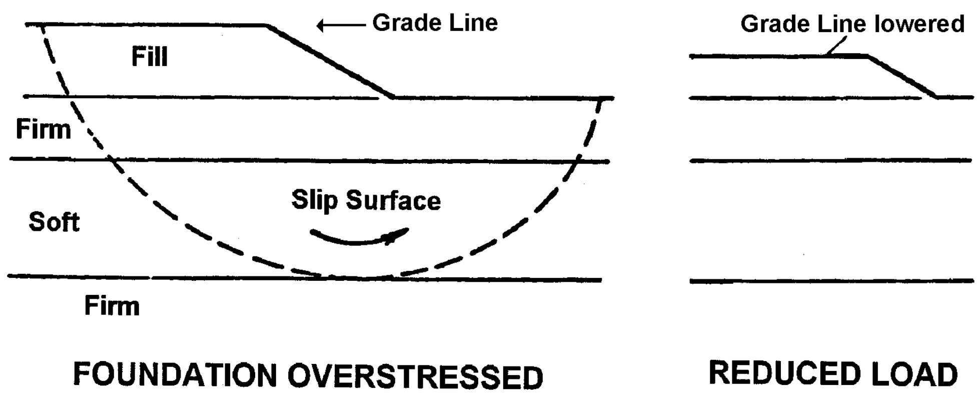



Figure 5-10 Reduction of Grade Line

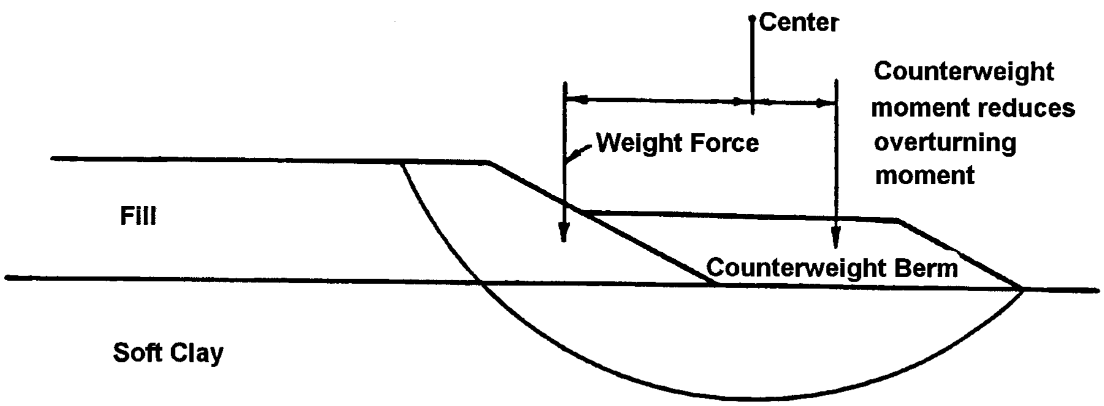



Figure 5-11 Use of Counterweight Berm to Improve Slope Stability

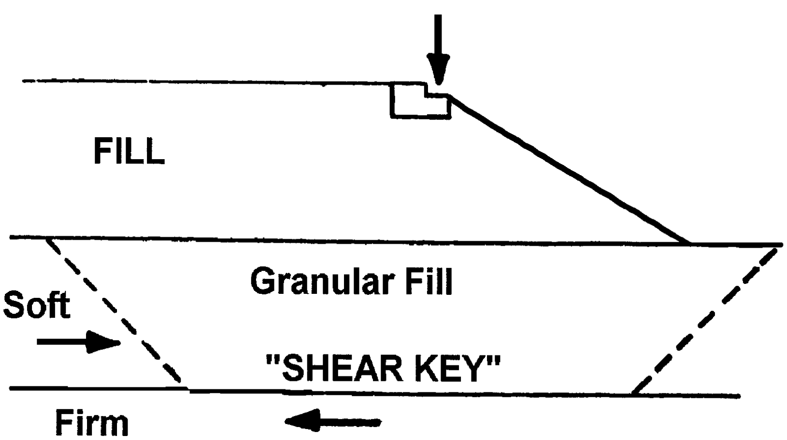



Figure 5-12 Use of Shear Key to Improve Slope Stability

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## 5.10 CUT SLOPE STABILITY

The two most common types of cut slope failures are deep-seated and shallow surface failures.

Type 1. Deep Seated Failure

Deep seated failure usually occurs in clay cut slopes. The clay has insufficient shearing strength to support the slope, and a circular arc shear failure occurs. If the clay has water bearing silt or sand layers, the seepage forces will also contribute to the instability. Figure 5-13 shows an example of a deep seated failure and a possible design solution.

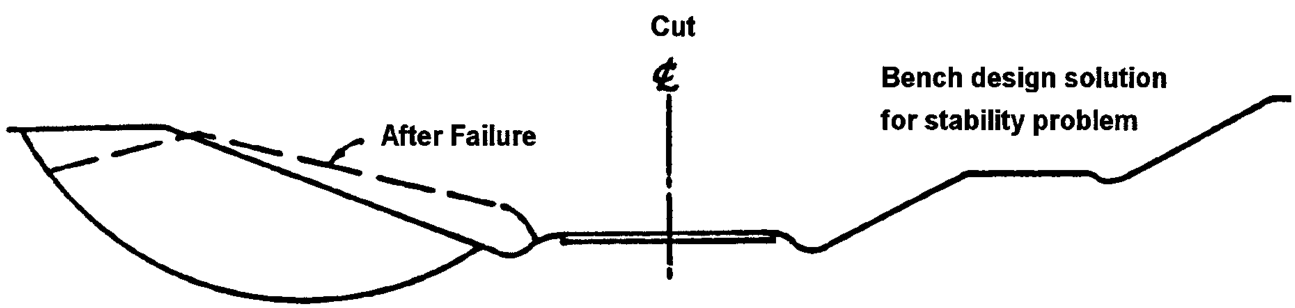



Figure 5-13: Deep Seated Slope Failure (Left) and Bench Slope Design (Right) to Prevent Slope Failure.

The following are typical design solutions to clay cut slope stability problems:

| | Design Solution | Effect on Stability |
|---|---|---|
| a. | Flatten slope. | Reduces overturning force. |
| b. | Bench slope. | Reduces overturning force. |
| c. | Buttress toe. | Increases resisting force. |
| d. | Lower water table. | Reduces seepage force. |

CAUTION: Design of cut slopes in clay should not be based on undrained strength of the clay from clay samples obtained before the cut is made. Designs based on undrained strength will be unconservative. The reason is that when the cut is made the effective stress is reduced because load is removed. This decrease in effective stress will allow the clay to swell and lose strength if the water is made available to the clay as illustrated as shown in Figure 5-14.

UNDRAINED CLAY IN CUT GRADUALLY WEAKENS AND MAY FAIL LONG AFTER CONSTRUCTION

Therefore, design of cut slopes in clays should be based on effective strength parameters so that the reduction in effective stress resulting from the cut excavation can be taken into account.

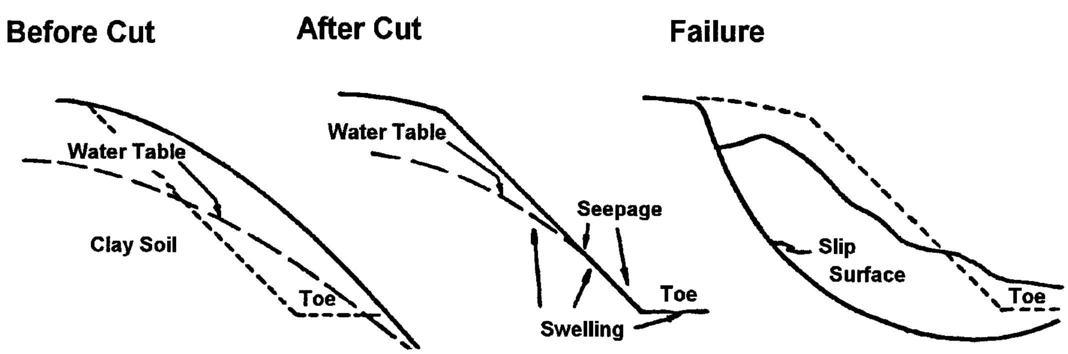



Figure 5-14: Typical Cut Slope Failure Mechanism in Clay Soils

Type 2. Surface Failures

Shallow surface failures (sloughs) are the most common clay or silt cut slope problem. These may involve either an entire slope or local areas in the slope.

The prime cause of shallow surface failures is water seepage. Water seepage reduces the strength of the surface soils, causing them to slide or flow. Soils most likely to be unstable are water bearing silts and layered clays.

Sloughing of slopes due to ground water seepage can often be remedied by placing a 2-3 foot thick rock or gravel blanket over the critical area. The blanket reduces the seepage forces, drains the water, and acts as a weight on the unstable soil. The blanket should be "keyed" into the ditch at the toe of slope. The key should extend about 4 feet below the ditch line and be about 4 feet wide. A geotextile should be placed both under the key and against the slope before blanket placement. Construction of the blanket should proceed from the toe upwards. The most effective placement is by a dozer which will track over and compact the lower blanket areas during placement of upper areas.

Factor of Safety - Cut Slopes

For stability of fine-grained cut slopes, current practice requires a minimum factor of safety against sliding of 1.50. The higher factor of safety for backslopes versus embankments is based upon the knowledge that cut slopes may deteriorate with time as a result of natural drainage conditions that embankments do not experience.

## 5.11 LATERAL SQUEEZE OF FOUNDATION SOIL

Field observations and measurements have shown that some bridge abutments supported on piling driven through thick deposits of soft compressible soils have tilted toward the backfill. Many of the structures have experienced large horizontal movements resulting in damage to the structure. The cause of this problem is the unbalanced fill load, which "squeezes" (consolidates) the soil laterally. This "lateral squeeze" of the soft foundation soil can transmit excessive lateral thrust which may bend or push the piles out, causing the abutment to rotate back toward the fill, as illustrated in Figure 5-18.

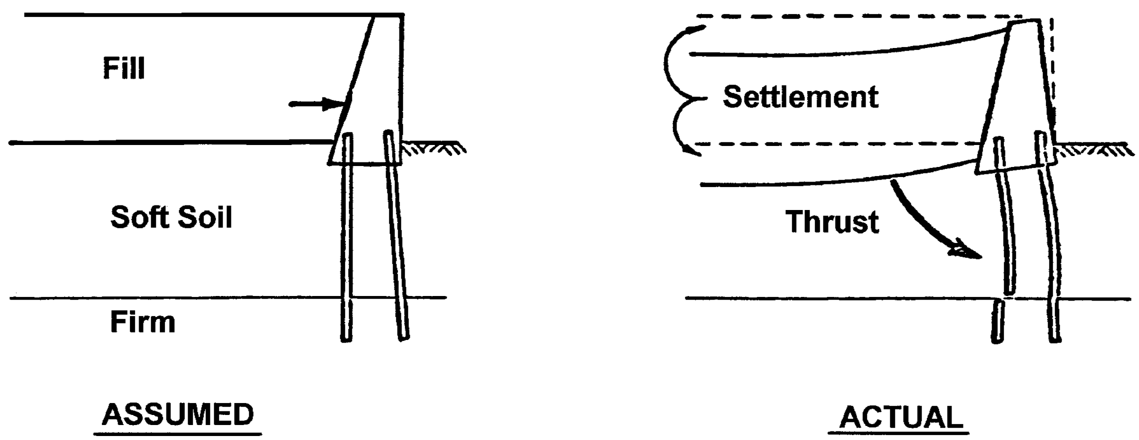



Figure 5-18: Lateral Squeeze Mechanism

### 5.11.1 Can Tilting Occur?

Experience has shown that if the applied surface load imposed by the fill weight exceeds 3 times the cohesive shear strength of the soft soil, i.e.,

If $\gamma_{Fill}$ x $H_{Fill} > 3C$

then this lateral squeeze of the foundation soil and abutment tilting can occur.

Therefore, using the above relationship, the possibility of abutment tilting can be evaluated in design. For all practical purposes, the fill unit weight can be assumed at 125 pcf. The cohesive strength C of the soft soil must be determined either from in situ field vane shear tests or triaxial tests on high quality undisturbed Shelby tube samples.

### 5.11.2 Estimation of Horizontal Abutment Movement

The amount of horizontal movement the abutment may undergo toward the fill can also be estimated in design. The following table contains case history information for nine structures where measurements of abutment movements have been made:

**SUMMARY OF ABUTMENT MOVEMENTS***

| Foundation | Fill Settlement (Inches) | Abutment Settlement (Inches) | Abutment Tilting (Inches) | Ratio of Abutment Tilting to Fill Settlement |
|---|---|---|---|---|
| Steel H-piles | 16 | Unknown | 3 | 0.19 |
| Steel H-piles | 30 | 0 | 3 | 0.10 |
| Soil bridge | 24 | 24 | 4 | 0.17 |
| Cast-in-place pile | 12 | 3.5 | 2.5 | 0.19 |
| Soil bridge | 12 | 12 | 3 | 0.25 |
| Steel H-piles | 48 | 0 | 2 | 0.06 |
| Steel H-piles | 30 | 0 | 10 | 0.33 |
| Steel H-piles | 5 | 0.4 | 0.5 to 1.5 | 0.1 to 0.3 |
| Timber Piles | 36 | 36 | 12 | 0.33 |

*Highway Research Record 334, 1971

This data provides a basis for estimating horizontal abutment movement for similar problems, providing a reasonable estimate of the post-construction fill settlement is made, using data from consolidation tests on high quality undisturbed Shelby tube samples. Note that the data for the structures listed in the previous summary showed horizontal abutment movement to range from 6 to 33 percent of the vertical fill settlement, with the average being 21 percent.

Therefore, if the fill load exceeds the 3C limit, then the horizontal abutment movement that may occur can reasonably be estimated as 25 percent of the vertical fill settlement, i.e.,

$$\text{Horizontal Abutment Movement} = 0.25 \times \text{Fill Settlement}$$

### 5.11.3 Design Solutions to Prevent Abutment Tilting

The best way to handle the abutment-tilting problem is to get the fill settlement out before the abutment piling are driven.

If the construction time schedule or other factors do not permit the settlement to be removed before the piling can be driven, then the problems resulting from abutment tilting can be mitigated by the following design provisions:

1. Use sliding plate expansion shoes large enough to accommodate the anticipated horizontal movement.

2. Make provisions to fill in the bridge deck expansion joint over the abutment by inserting either metal plate fillers or larger neoprene joint fillers.

3. Design piles for downdrag forces due to settlement.

4. Use steel H-piles for the abutment piling since steel H-piles are capable of taking large tensile stresses without failing.

5. Use backward battered piles at the abutment and particularly the wingwalls.

Movements should also be monitored so that predicted movement can be compared to actual.

## 5.12 APPLE FREEWAY DESIGN EXAMPLE – SLOPE STABILITY

In this chapter the Apple Freeway Example Problem is used to illustrate the analysis and design of an embankment with respect to stability consideration. Slope stability analysis using the Normal Method by hand calculations is performed and compared to computer generated solutions. A sliding block analysis is performed and the possibility of lateral squeeze is also examined.

| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | $P_o$ Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| → | **Slope Stability** |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

Design Soil Profile
Circular Arc
Analysis Sliding Block
Analysis Lateral Squeeze

Apple Freeway Design Example – Slope Stability
Exhibit A

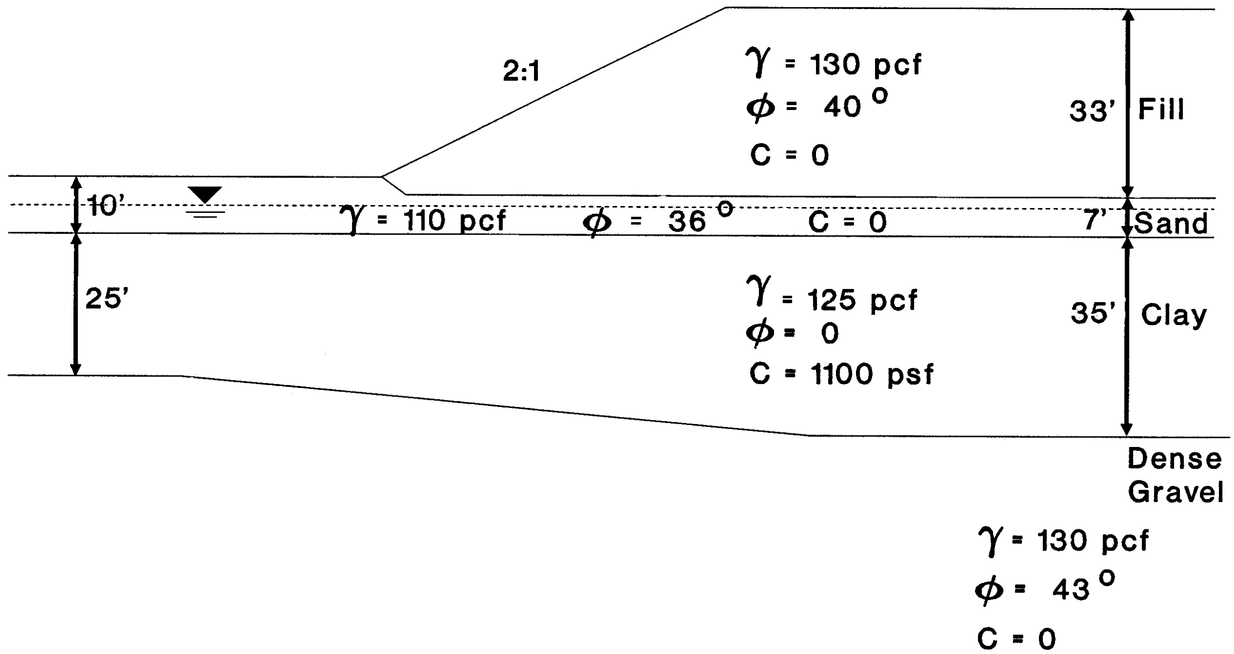
**Given:** The proposed embankment geometry (Figure 2-5) and soil properties at the east approach of the Apple Freeway Bridge. Assume that the shallow (≈ 3′) surface layer of organic has been removed and replaced with select material.

**Required:** Compute the embankment stability with respect to circular arc failure, sliding block failure and lateral squeeze.

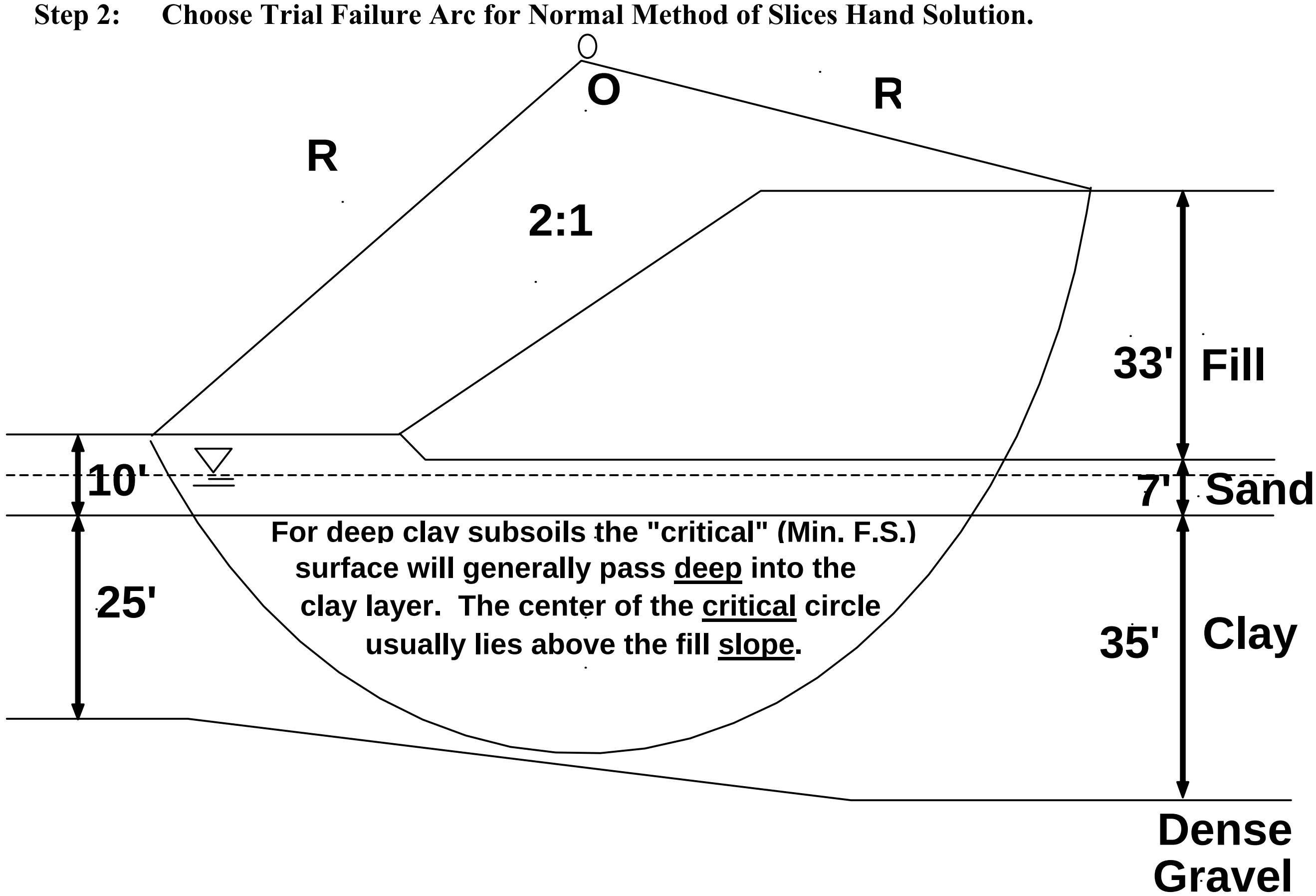
**Solution:**

- **Compute F.S. against circular arc failure (Normal Method/ Hand Solution) and check with computer solution**
- **Compute F.S. against circular arc failure by the Bishop Simplified Method**
- **Compute F.S. against sliding block failure using Rankine block analysis**
- **Check if lateral squeeze is possible at this embankment location**

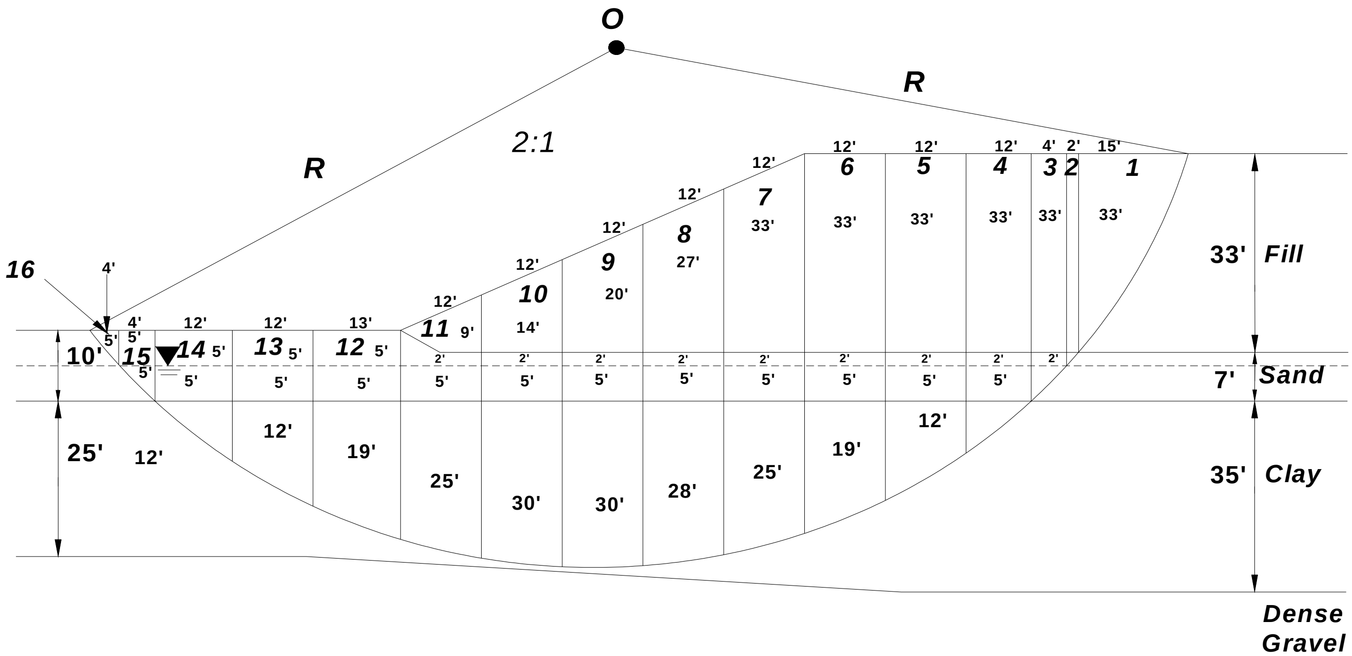
**Step 1:** **Obtain Soil Profile and Design Parameters**



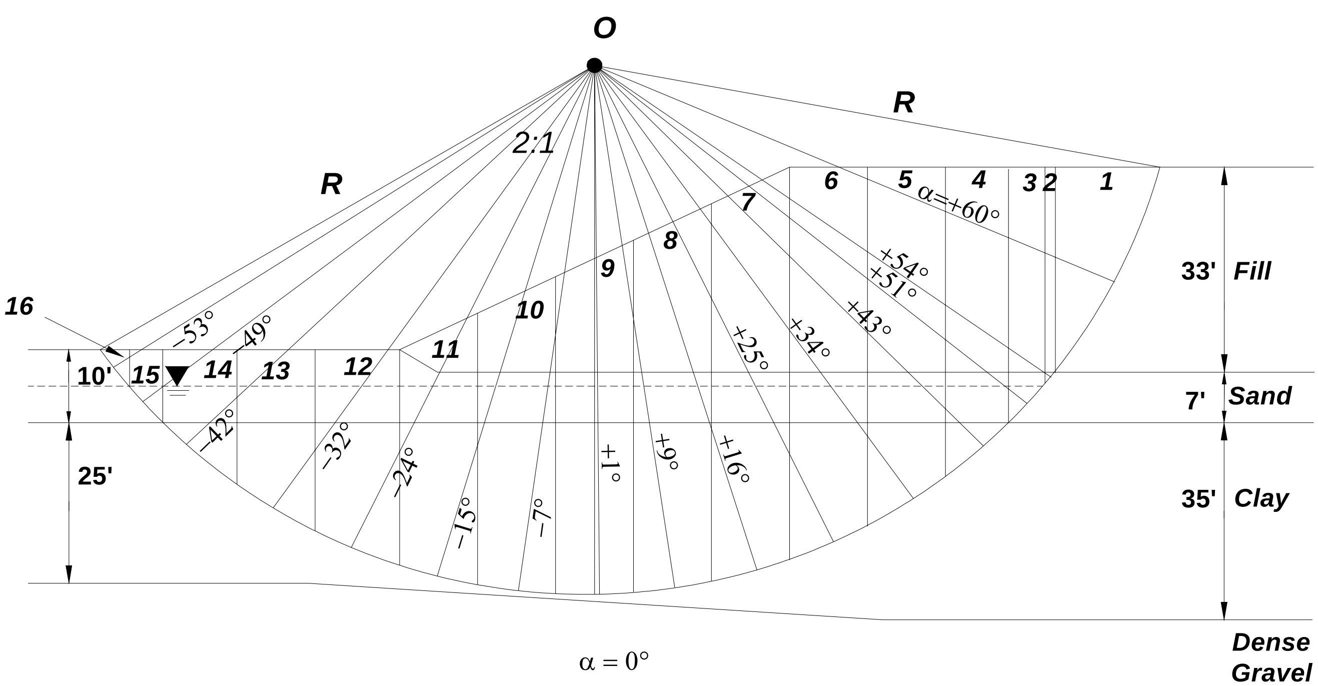

**Step 2:** **Choose Trial Failure Arc for Normal Method of Slices Hand Solution.**



**Step 3: Circular Arc Analysis – Divide Mass Above Failure Surface into Vertical Slices.**

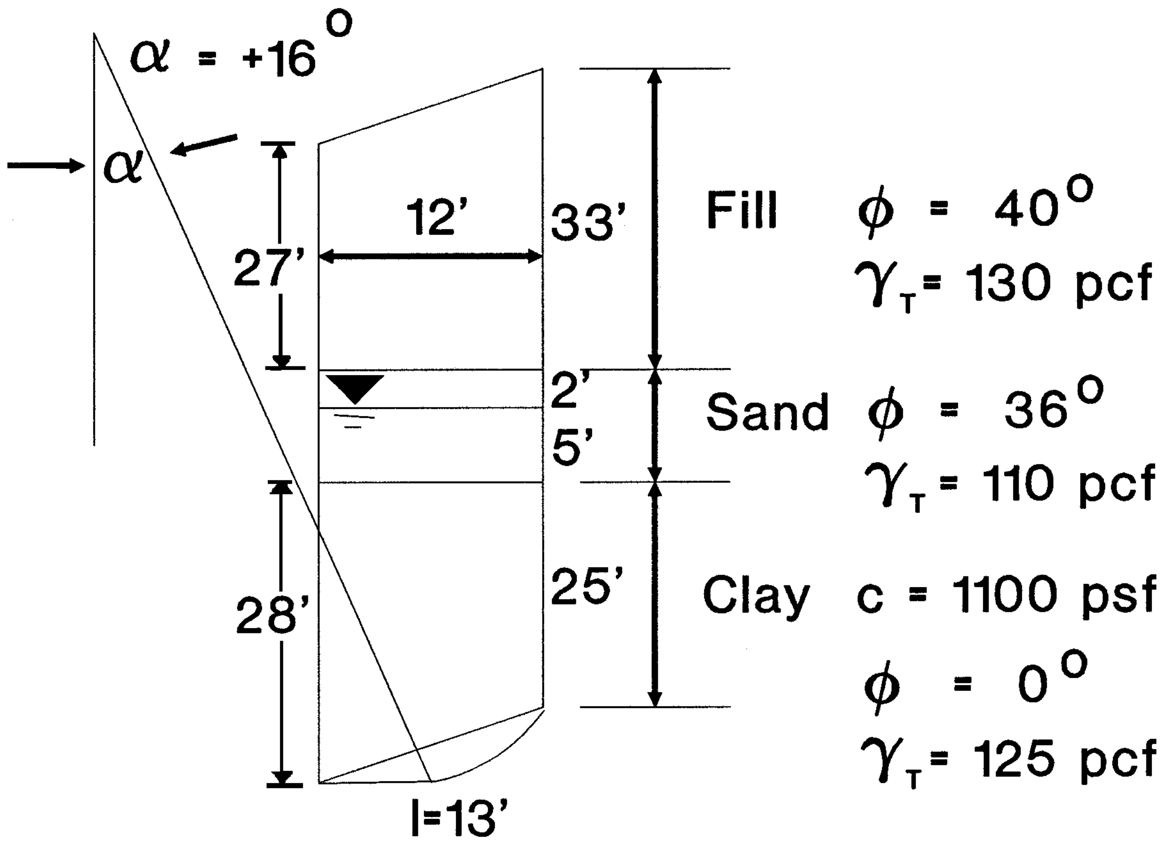



**Step 4: Determine α Angles.**



**Step 5: Compute Resisting and Driving Forces for All Slices.**

Workshop Design Problems Example Computation Slice 7




$$W_T = (12)\left(\frac{27+33}{2}\right)(130) + (12)(7)(110) + (12)\left(\frac{28+25}{2}\right)(125) = 95{,}790^{\#}$$

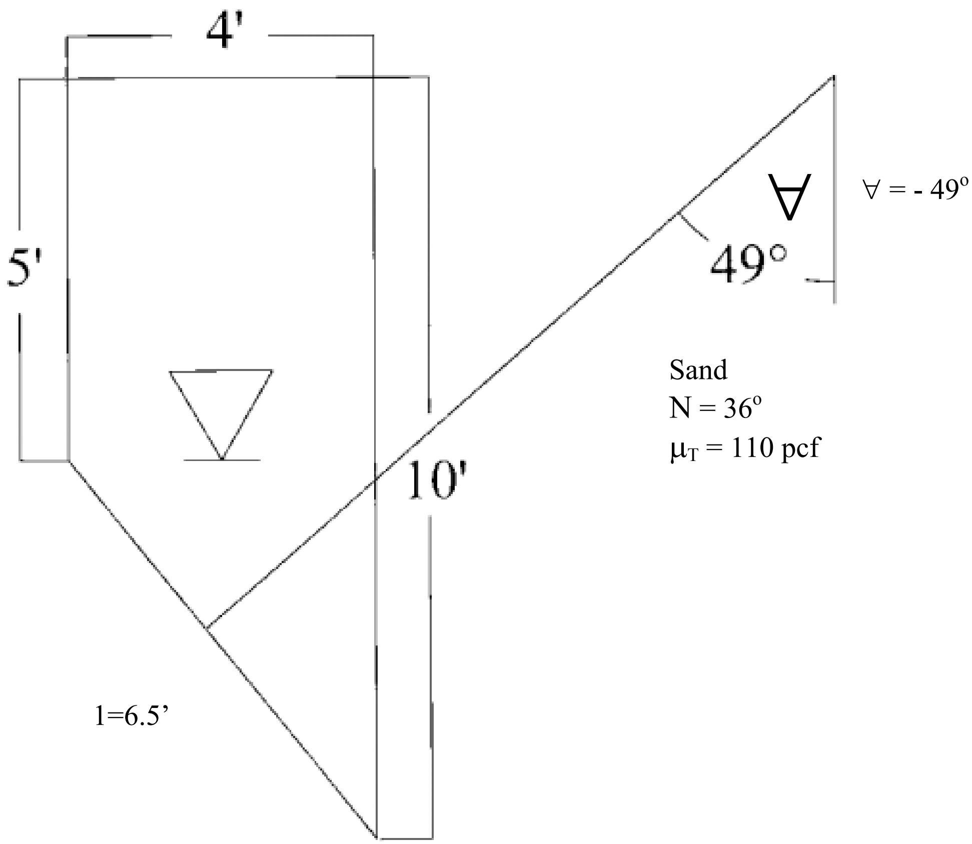
$T = W_T \text{ Sin } \alpha = 95{,}790^{\#} (\text{Sin } 16°) = 26{,}403^{\#}$

Bottom of Slice is in Clay where $\phi = 0 \rightarrow N \text{ Tan } \phi = 0$

$c\ l = (1100)(13) = 14{,}300^{\#}$

For slice 7: $T = 26{,}403^{\#}$ (Driving Force)
$c\ l = 14{,}300^{\#}$ (Resisting Force)
$N \text{ Tan } \phi = 0$, Since $\phi = 0$

Workshop Problems Example Computation Slice 15




$$W_T = (4)\left(\frac{10+5}{2}\right)(110) = 3{,}300^{\#}$$

$$T = W_T \text{ Sin}\alpha = 3{,}300^{\#}(\text{Sin} - 49°) = -2{,}491^{\#}$$

Note: T is negative for this slice since the weight tends to RESIST sliding.

Bottom of slice is in sand with $\phi = 36°$
$c = 0 \rightarrow cl = 0$

$$N = W_T \text{ Cos}\alpha - \mu l$$

$$= (3{,}300^{\#})(\text{Cos} - 49°) - (\frac{5}{2})(60)(6.5)$$

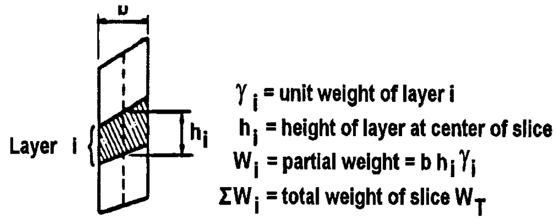
$$= 2{,}165^{\#} - 975^{\#} = 1{,}190^{\#}$$

N Tan $\phi$ = $1{,}190^{\#}$ (Tan 36°) = $865^{\#}$

For slice 15: T = $-2{,}491^{\#}$ (Driving Force)
N Tan$\phi$ = $865^{\#}$ (Resisting Force)
cl = 0, Since c = 0

## Step 6: Compute Weights for Each Slice.

Tabular Form for Computing Weights of Slices



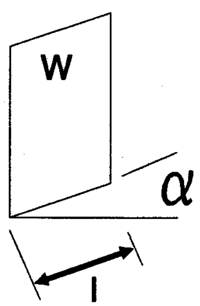

| Slice No. | B | $h_I$ | $\gamma_i$ | $W_i$ | $\sum W_i = W_T$ |
|---|---|---|---|---|---|
| 1 | 15 | 33/2 | 130 | 32175 | 32175 |
| 2 | 2 | 33 | 130 | 8580 | |
| | | 2/2 | 110 | 220 | 8800 |
| 3 | 4 | 33 | 130 | 17160 | |
| | | (7+2)/2 | 110 | 1980 | 19140 |
| 4 | 12 | 33 | 130 | 51480 | |
| | | 7 | 110 | 9240 | |
| | | 12/27 | 125 | 9000 | 69720 |
| 5 | 12 | 33 | 130 | 51480 | |
| | | 7 | 110 | 9240 | |
| | | (19+12)/2 | 125 | 23250 | 83970 |
| 6 | 12 | 33 | 130 | 51480 | |
| | | 7 | 110 | 9240 | |
| | | (19+25)/2 | 125 | 33000 | 93720 |
| 7 | 12 | (27+33)/2 | 130 | 46800 | |
| | | 7 | 110 | 9240 | |
| | | (25+28)/2 | 125 | 39750 | 95790 |
| 8 | 12 | (20+27)/2 | 130 | 36660 | |
| | | 7 | 110 | 9240 | |
| | | (36+28)/2 | 125 | 43500 | 89400 |
| 9 | 12 | (14+20)/2 | 130 | 26520 | |
| | | 7 | 110 | 9240 | |
| | | 30 | 125 | 45000 | 80760 |
| 10 | 12 | (9+14)/2 | 130 | 17940 | |
| | | 7 | 110 | 9240 | |
| | | (28+30)/2 | 125 | 43500 | 70680 |
| 11 | 12 | (9+3)/2 | 130 | 9360 | |
| | | 7 | 110 | 9240 | |
| | | (25+28)/2 | 125 | 39750 | 58350 |
| 12 | 13 | 10 | 110 | 14300 | |
| | | (19+25)/2 | 125 | 35750 | 50050 |
| 13 | 12 | 10 | 110 | 13200 | |
| | | (12+19)/2 | 125 | 23250 | 36450 |
| 14 | 12 | 10 | 110 | 13200 | |
| | | 12/2 | 125 | 9000 | 22200 |
| 15 | 4 | (5+10)/2 | 110 | 3300 | 3300 |
| 16 | 4 | 5/2 | 110 | 1100 | 1100 |

Workshop Design Problem

**Step 7: Compute Factor of Safety.**

Tabular Form for Calculating FS by Normal Method of Slices.

| Slice No. | $W_T$ (lb) | l (ft) | $\alpha$ (deg) | C (psf) | $\phi$ (deg) | $\mu$ (psf) | $\mu l$ (lb) | $W_T$ Cos ∀ (lb) | N (lb) | N TanN (lb) | Cl (lb) | T (lb) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 32,175 | 36 | 60 | 0 | 40 | 0 | 0 | 16,088 | 16,088 | 13,499 | 0 | 27,864 |
| 2 | 8,800 | 3 | 54 | 0 | 36 | 0 | 0 | 5,173 | 5,173 | 3,758 | 0 | 7,119 |
| 3 | 19,140 | 7 | 51 | 0 | 36 | 150 | 1050 | 12,045 | 10,995 | 7,988 | 0 | 14,875 |
| 4 | 62,720 | 17 | 43 | 1100 | 0 | - | - | - | - | 0 | 18,700 | 47,549 |
| 5 | 83,790 | 15 | 34 | 1100 | 0 | - | - | - | - | 0 | 16,500 | 46,955 |
| 6 | 93,720 | 15 | 25 | 1100 | 0 | - | - | - | - | 0 | 16,500 | 39,608 |
| 7 | 95,790 | 13 | 16 | 1100 | 0 | - | - | - | - | 0 | 14,300 | 26,403 |
| 8 | 89,400 | 13 | 9 | 1100 | 0 | - | - | - | - | 0 | 14,300 | 13,985 |
| 9 | 80,760 | 12 | 1 | 1100 | 0 | - | - | - | - | 0 | 13,200 | 1,409 |
| 10 | 70,680 | 12 | -7 | 1100 | 0 | - | - | - | - | 0 | 13,200 | -8,614 |
| 11 | 59,350 | 13 | -15 | 1100 | 0 | - | - | - | - | 0 | 14,300 | -15,102 |
| 12 | 50,050 | 14 | -24 | 1100 | 0 | - | - | - | - | 0 | 15,400 | -20,357 |
| 13 | 36,450 | 14 | -32 | 1100 | 0 | - | - | - | - | 0 | 15,400 | -19,316 |
| 14 | 22,200 | 16 | -42 | 1100 | 0 | - | - | - | - | 0 | 17,600 | -14,855 |
| 15 | 3,300 | 6.5 | -49 | 0 | 36 | 150 | 975 | 2,165 | 1,190 | 865 | 0 | -2,491 |
| 16 | 1,100 | 6.5 | -53 | 0 | 36 | 0 | 0 | 662 | 662 | 481 | 0 | -878 |
| | | | | | | | | | $\sum$ | 26,591 | 169,400 | 144,154 |



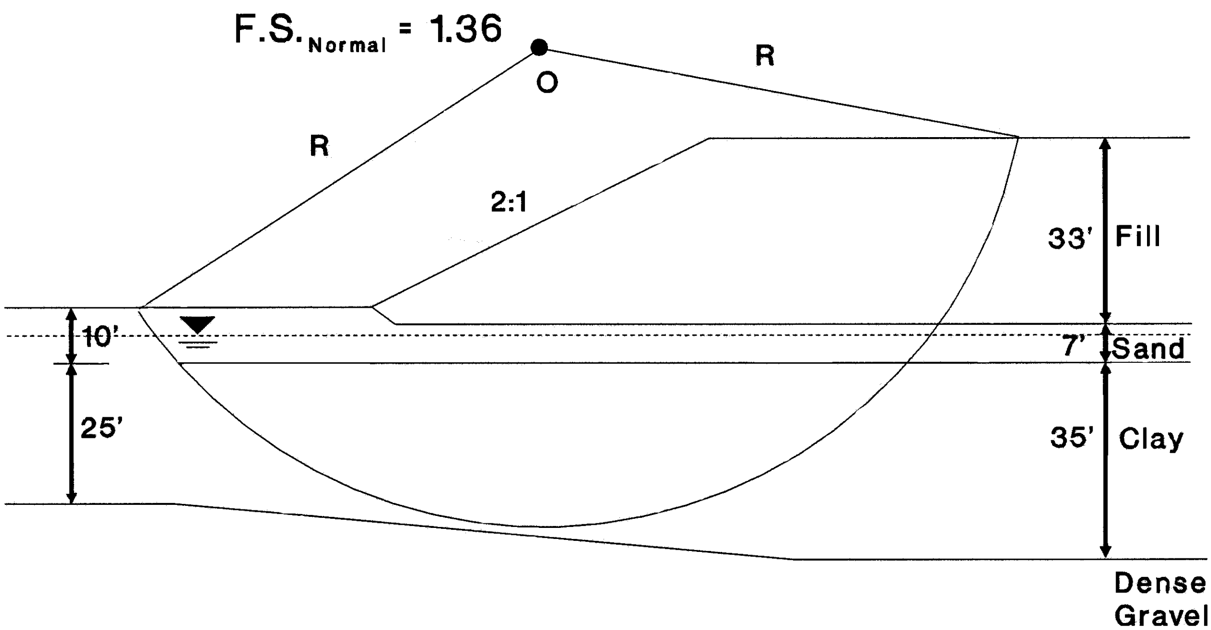

C = cohesion intercept
$\phi$ = friction angle
$\mu$ = pore pressure
$W_T$ = total wt. of slice (soil + water)

$$F = \frac{\Sigma (W_T \cos\alpha - \mu l)\tan\phi + \Sigma cl}{W_T \sin\alpha}$$

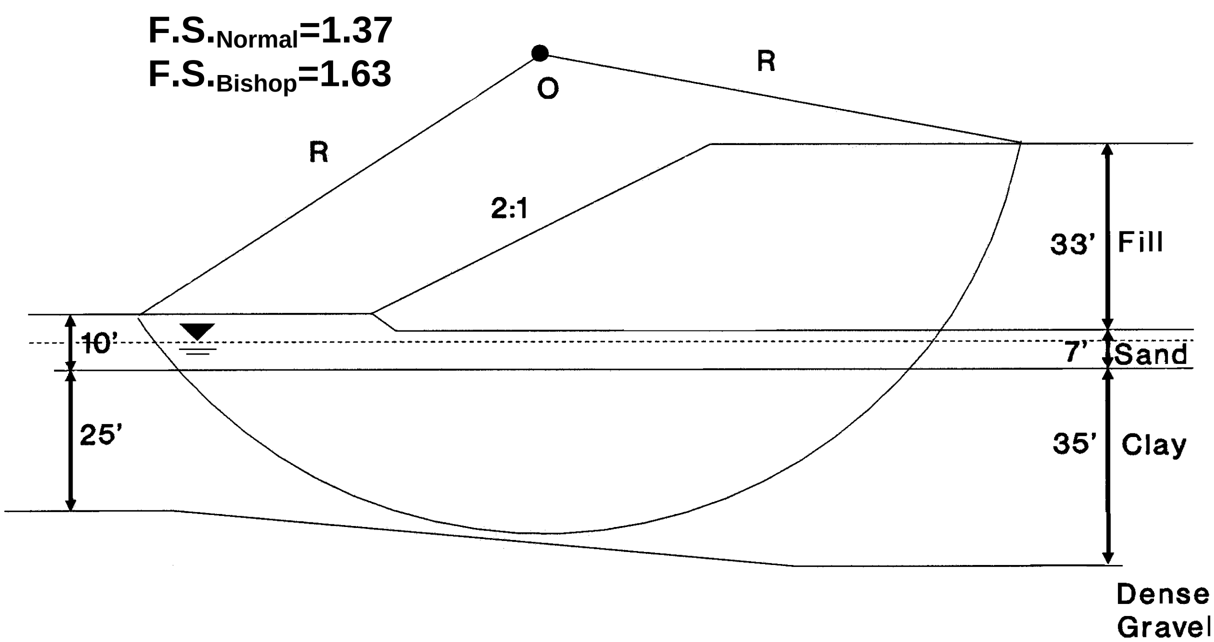
$$F = \frac{\Sigma N \tan\phi + \Sigma cl}{\Sigma T} = \frac{26,591 + 169,400}{144,154} = \underline{1.36}$$

TABULAR FORM FOR CALCULATING F.S. BY NORMAL METHOD OF SLICES

Workshop Design Problem – Hand Solution

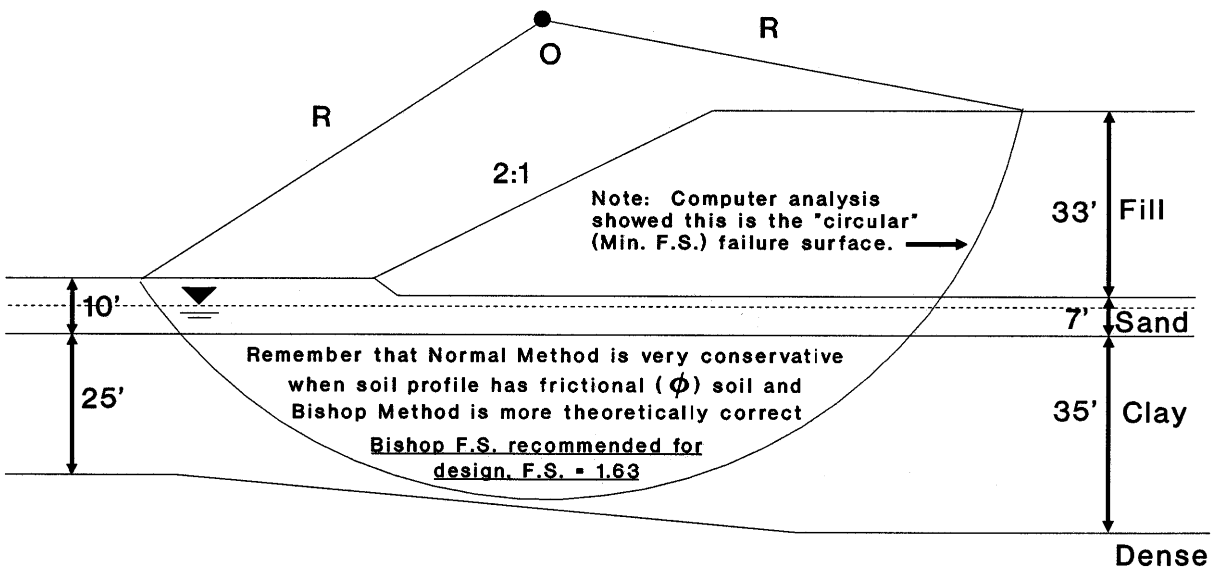



Workshop Design Problem – Computer Solution



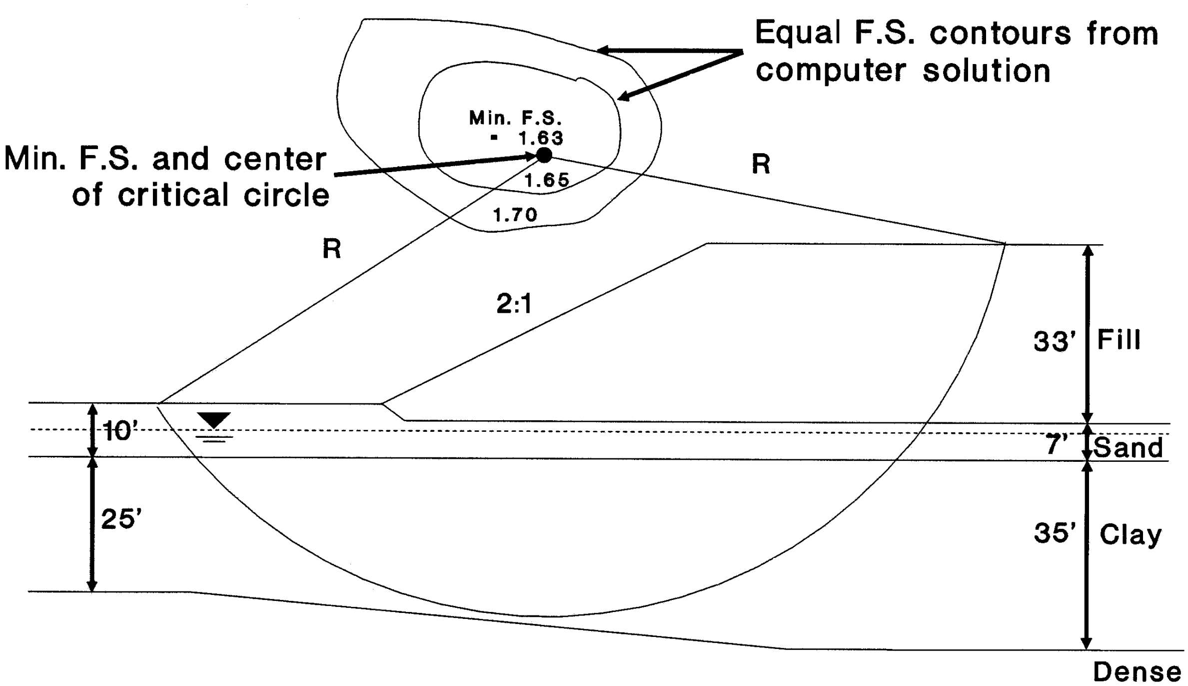
Comparison of Factors of Safety

**F.S. = 1.36 Normal Method - Hand Solution**
**F.S. = 1.37 Normal Method - Computer Solution**
**F.S. = 1.63 Bishop Method - Computer Solution**



Workshop Design Problem

For Design use Min. F.S. (Bishop) = 1.63

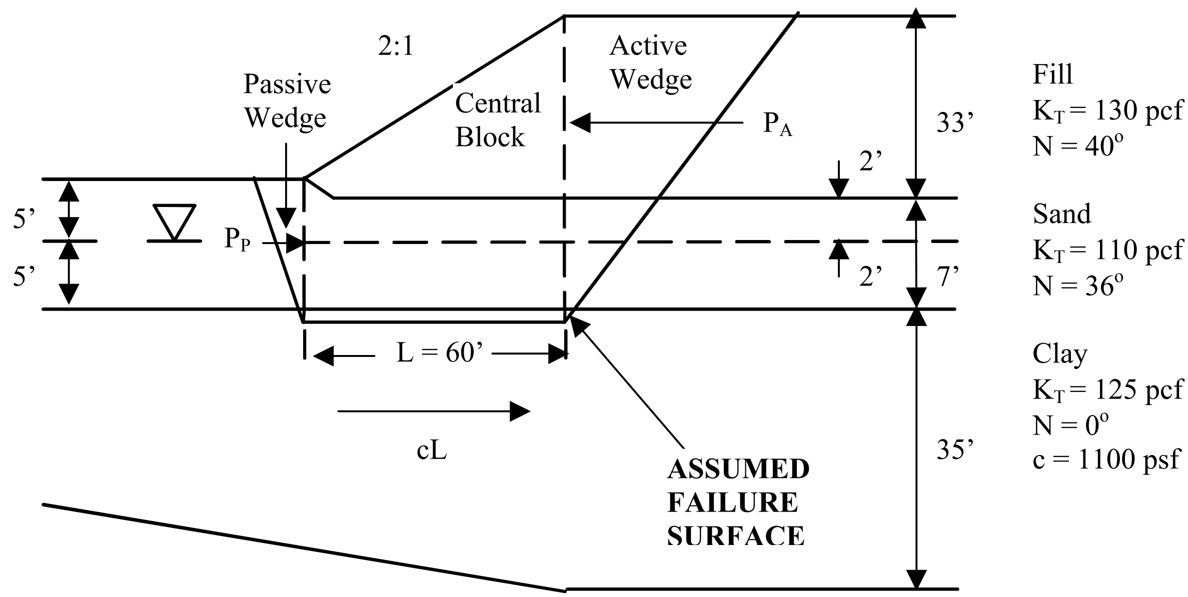


Workshop Design Problem

**WORKSHOP DESIGN PROBLEM – SLIDING BLOCK ANALYSIS**

Compute Factor of Safety against sliding block type failure along top of clay layer for assumed failure surface shown.

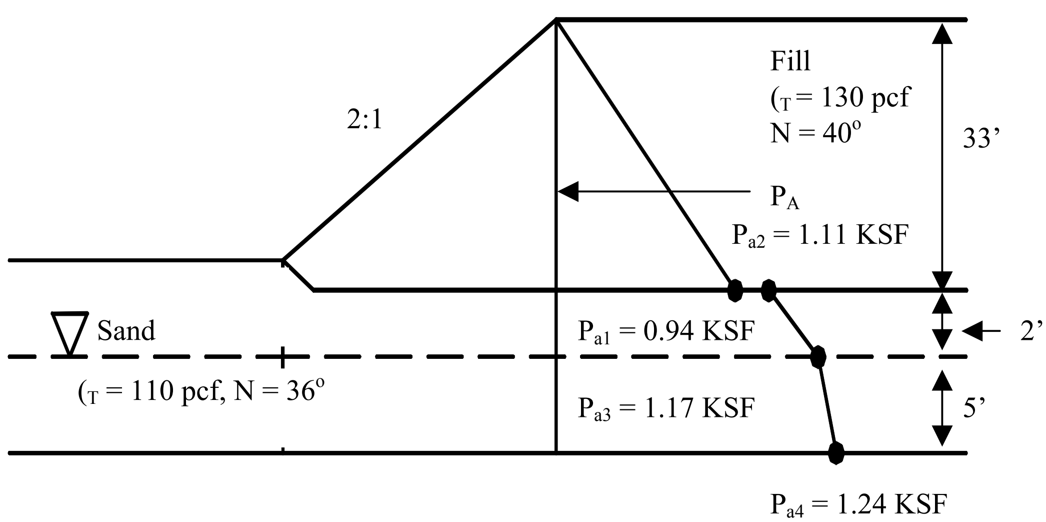
**Step 1: Choose Trial Failure Surface.**




**Step 2: Compute Active Force ($P_A$)**

Fill = Soil Layer 1; Fill $\phi = 40°$; $K_{A1} = \tan^2(45° - 40°/2) = \tan^2(25°) = 0.22$

Soil Layer 2; Sand $\phi = 36°$; $K_{A2} = \tan^2(45° - 36°/2) = \tan^2(27°) = 0.26$



**Step 3: Compute Active Pressure.**

$p_{a1}$ (base of fill) = $\gamma_1 h_1 K_{A1}$ = (0.130 kcf)(33′)(0.22)=0.94 ksf

$p_{a2}$ (top of sand) = $\gamma_1 h_1 K_{A2}$ = (0.130 kcf)(33′)(0.26)=1.11 ksf

$p_{a3}$ (2' below top of sand*) = 1.11 ksf+(0.110 kcf)(2′)(0.26)=1.17 ksf
(*Water table elevation)

$p_{a4}$ (base of sand layer) = 1.17 ksf+(0.050 kcf*)(5′)(0.26)=1.24 ksf
(*Buoyant weight below water table)

**Step 4: Plot Active Pressure Diagram & Compute Active Force.**

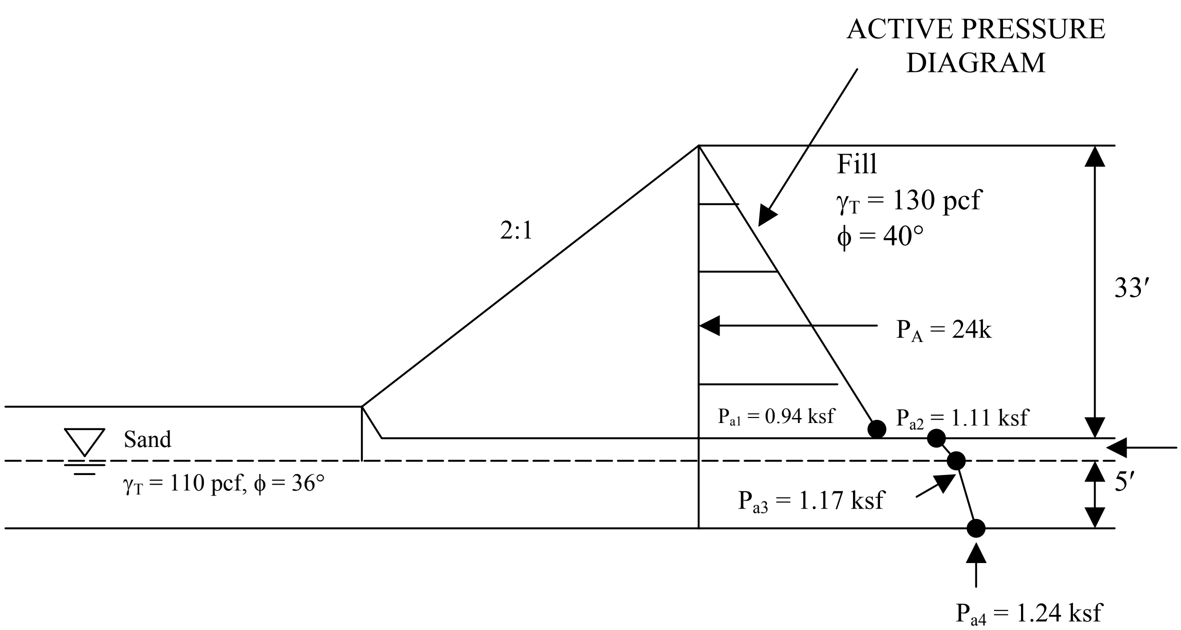
$P_A$ = Active Force = Area of Pressure Diagram (per ft.)

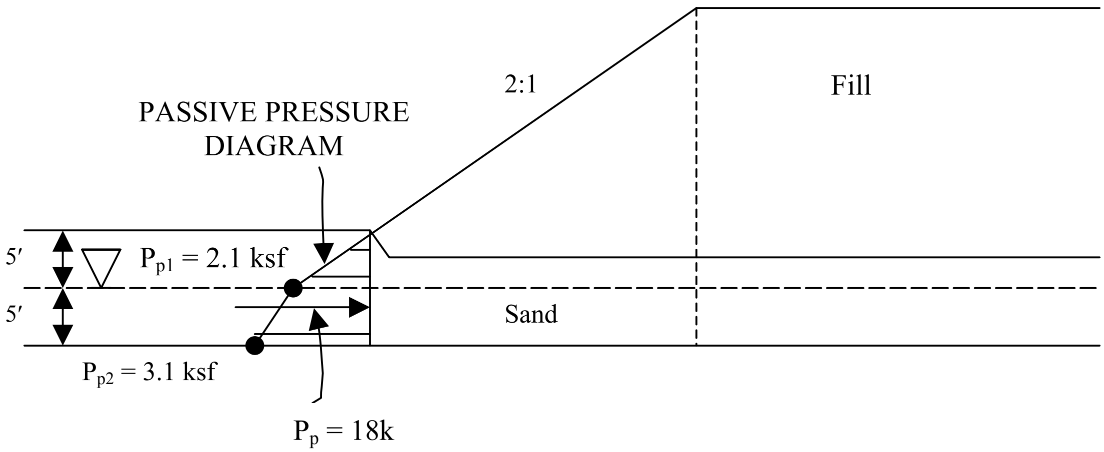
$\therefore P_A$ = (0.94 ksf)(33′)(1/2)(1′)

+ ((1.11 ksf + 1.17 ksf)/2)(2′)(1′)

+ ((1.17 ksf + 1.24 ksf)/2)(5′)(1′)

$= 15.5^K + 2.3^K + 6^K \therefore P_A \approx 24^K$






**Step 5: Compute Passive Force $P_P$.**

(a) Compute Passive Pressure

Sand $\phi$= 36°; $K_P$=$Tan^2$ (45°+$\phi$/2)=$Tan^2$ (45°+36°/2)=3.8

$p_{p1}$ (5′ below top of sand*) = (0.110 kcf)(5′)(3.8)=2.1 ksf (*At water table)

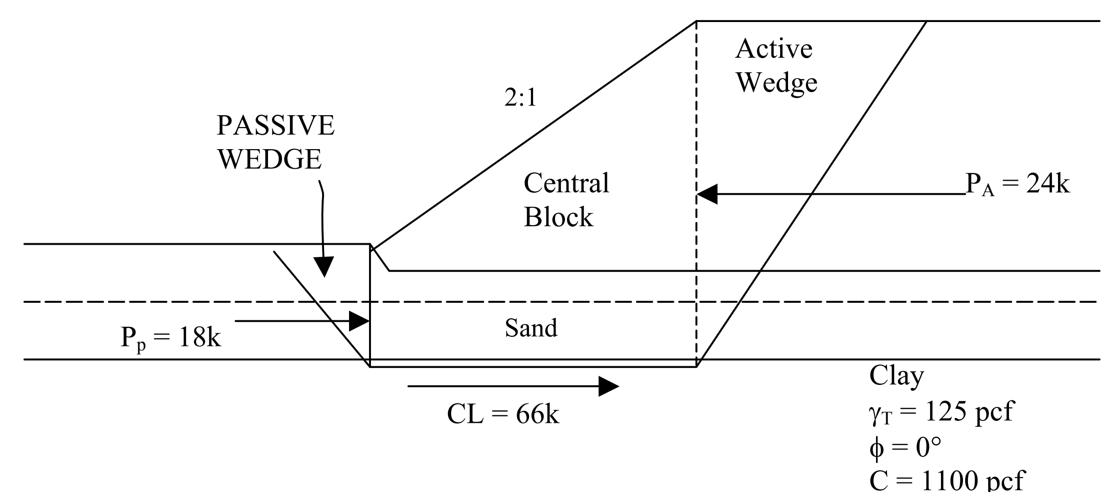
$p_{p2}$ (base of sand layer) = 2.1 ksf+(0.050 kcf*)(5′)(3.8)=3.1 ksf (*Buoyant weight below water table)

**Step 6: Plot Passive Pressure Diagram & Compute Passive Force.**

$\therefore P_P$ (per ft)= (2.1 ksf)(5′)(1/2)(1′)

+ ((2.1 ksf+3.1 ksf)/2)(5′)(1′)

= $5.3^K + 13^K \therefore P_P \approx 18^K$



**Step 7: Compute Resisting Force of Central Block.**

Assumed failure plane is along top of clay
C = 1100 psf = 1.1 ksf
L = 60′
$\therefore$ CL = (1.1ksf)(60′)(1′) = $66^K$ (per ft)

**Step 8: Compute Factor of Safety.**

$$\text{F.S.} = \frac{\text{Horizontal Resisting Forces}}{\text{Horizontal Driving Forces}} = \frac{P_p + CL}{P_A}$$

$$= \frac{18^K + 66^K}{24^K} = \frac{84^K}{24^K} = 3.5$$

F.S. = 3.5 OK $\therefore$ Circular Arc Failure More Critical

**CHECK FOR - LATERAL SQUEEZE**

Lateral Squeeze of Clay

Lateral squeeze causes pile supported abutments to rotate into embankment or spread footing abutments to move laterally.

Lateral Squeeze occurs if:

$$\gamma_{Fill}\, H_{Fill} > 3 \times \text{Cohesion}$$

For East Abutment:

130 pcf x 30' > 3 x 1100 psf
3900 psf > 3300 psf

∴ -can get lateral squeeze
-consider waiting period to dissipate settlement of fill
-do not construct abutments until settlement dissipates
(U=90%)

**Summary of the Approach Embankment Stability Phase for the Apple Freeway Design Problem**

- Design Soil Profile

  Soil layer unit weights and strength estimated.

- Circular Arc Analysis

  Approach embankment safety factor 1.63 against circular failure.

- Sliding & Block Analysis

  Approach embankment safety factor 3.5 against sliding failure.

- Lateral Squeeze

  Possible abutment rotation problem.

# CHAPTER 6.0
# EMBANKMENT SETTLEMENT

Embankment settlement is the most prevalent foundation problem in highway construction. Unlike stability problems, the results are seldom catastrophic but the cost of perpetual maintenance of continuing settlement are immense. The difficulty in preventing these problems is not as much a lack of technical expertise as a lack of communication between personnel involved in the roadway design and those involved in the structure design.

## 6.1 TYPICAL EMBANKMENT SETTLEMENT PROBLEMS

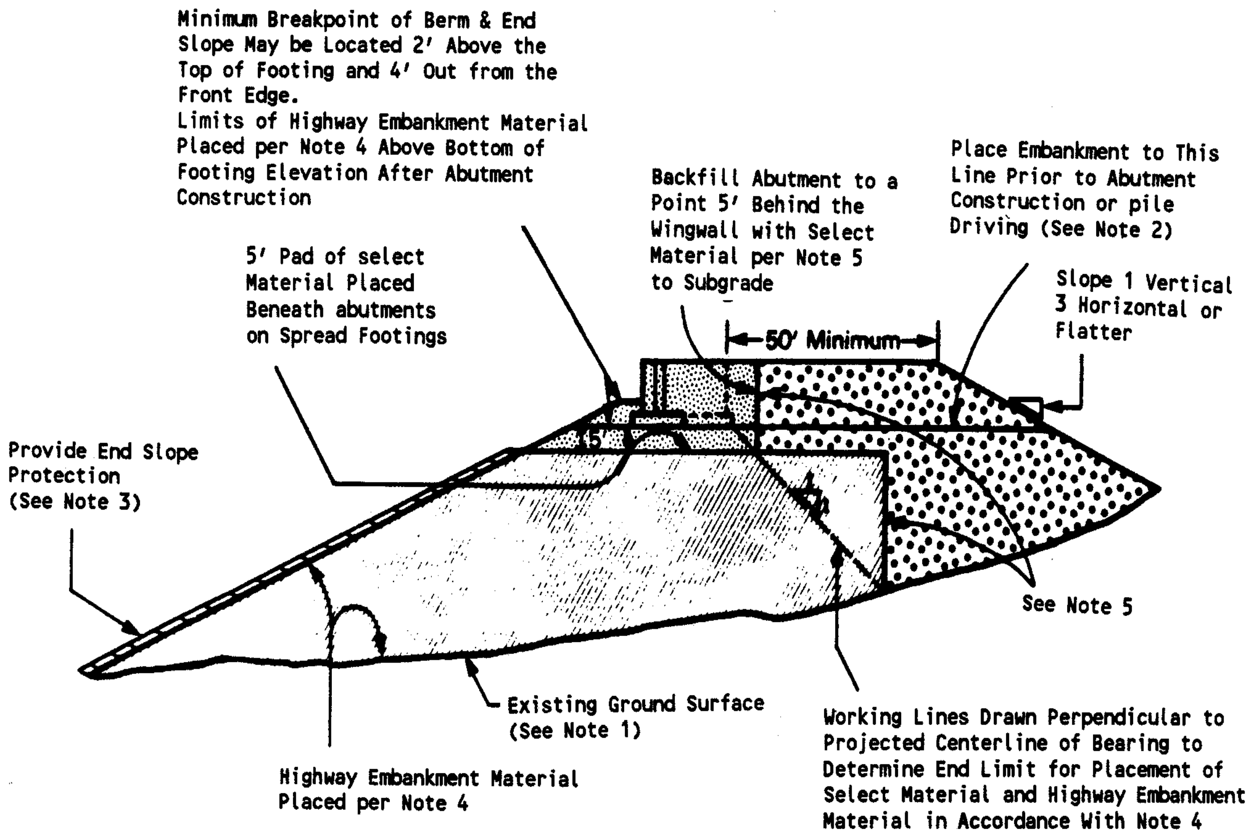
The design of a roadway embankment can utilize a wide range of soil materials and permit substantial amounts of settlement without affecting the performance of the highway. Roadway designers necessarily permit such materials to reduce project costs by utilizing cheap locally available soils. Structures are necessarily designed for little or no settlement to maintain specified highway clearances and to insure integrity of structural members. The approach embankment must affect a transition between roadway and structure while providing adequate structural foundation support. In most agencies the responsibility for approach embankment design is not defined as a structural issue, which results in roadway criteria being used across the structure. This is wrong; the approach embankment requires special materials and placement criteria to prevent internal consolidation and to moderate external consolidation.

## 6.2 COMMON DESIGN SOLUTIONS TO EMBANKMENT SETTLEMENT

### 6.2.1 Eliminate settlement within the approach embankment

A well constructed soil embankment, using quality control with regard to material and compaction, will not consolidate. Standard specifications and construction drawings should be prepared for the approach embankment area (normally designated to extend 50 feet behind the wingwall). The structural designer should have the responsibility for selecting the appropriate approach embankment cross section depending on selection of structure foundation type. A typical suggested approach embankment cross section is shown on Figure 6-1 for spread footing and pile foundations.

Special attention must be given to the interface area between the structure and the approach embankment, as this is where the famous "bump at the end of the bridge" occurs. The reasons for the bump are twofold; poor compaction of embankment material near the structure and migration of fine soil into drainage material. Poor densification is caused by restricted access of standard compaction equipment. Proper densification can be achieved by optimizing the soil gradation in this area to permit maximum density with minimum effort. Figure 6-2 shows a suggested detail for placement of drainage material. Typical specifications for select structure backfill and underdrain filter material to prevent the problem are included in Appendix E and F respectively. Similar results can be obtained by the use of prefabricated geocomposite drains which are attached to the backwall and connected to an underdrain.




Note 1: Topsoil shall be stripped beneath approach embankments less than 20′ in height from a rectangular or trapezoidal area abounded by lines 15 feet outside the abutment and wingwall footings, or to the top of slope, whichever is less. The depth or stripping shall be determined by the soils Engineer and displayed on the highway cross sections by the Design Engineer.

Note 2: At some sites, fill is to be placed to the subgrade of the roadway and allowed to stand, in order to consolidate underlying material, before piles are driven.

Note 3: Slope protection treatment shall be as specified by the Bridge Engineer.

Note 4: Highway embankment material placed within these limits shall have a maximum dimension of 6 inches and shall be compacted to 95% of AASHTO T-180 maximum density. Quantity to be included in highway estimate.

Note 5: Highway embankment material and select material shall be placed concurrently on both sides of the vertical payment line.

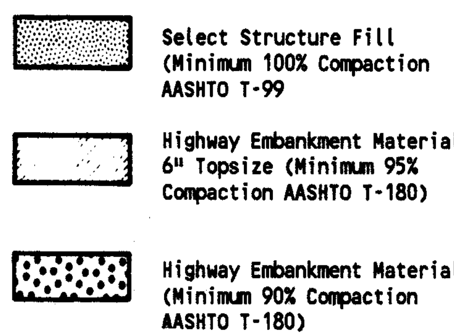



Figure 6-1: Suggested Approach Embankment Details

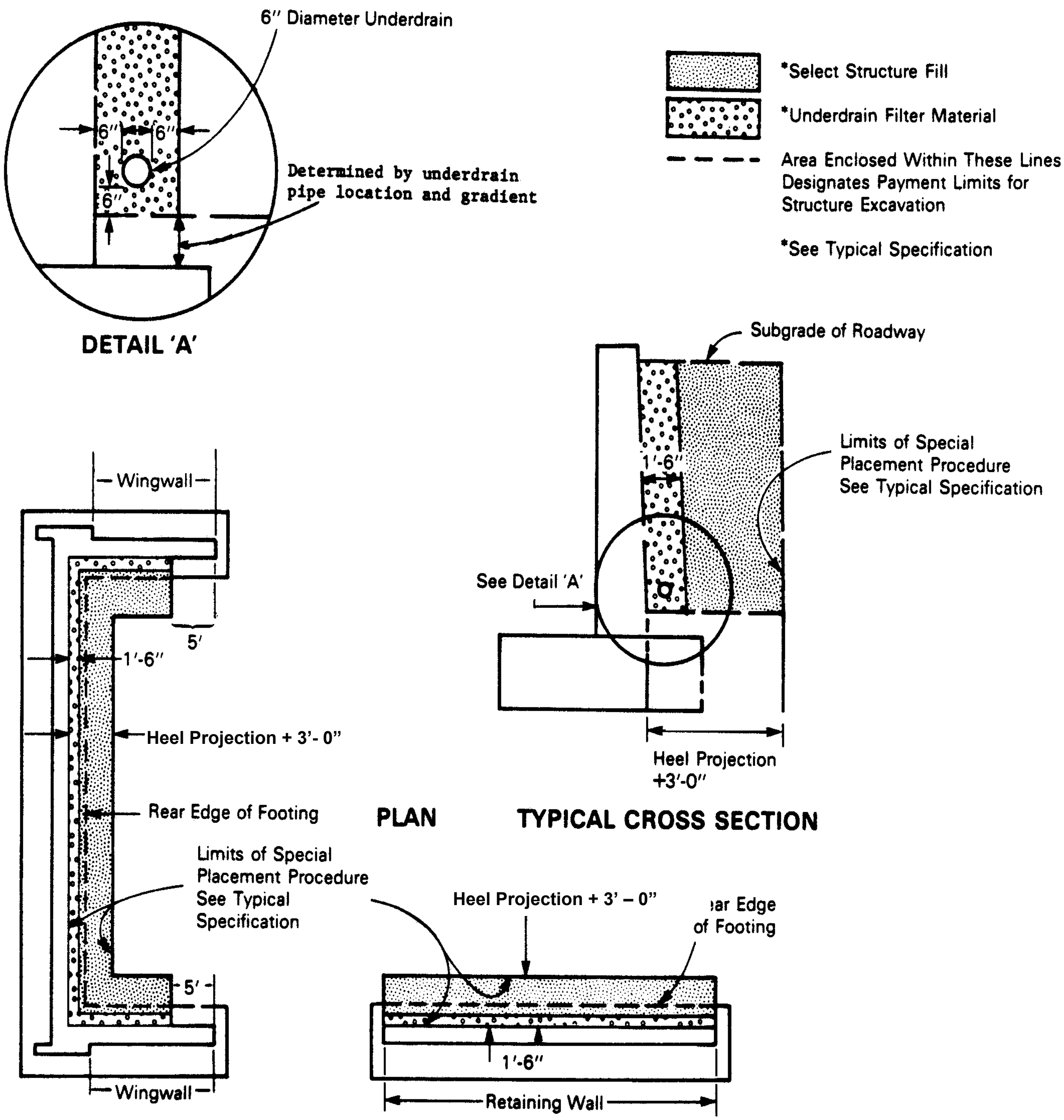



Figure 6-2: Structure backfill placement limits for porous drainage aggregate.

### 6.2.2 General Consideration for Select Structure Backfill

Select structure backfill is usually placed in relatively small quantities and in relatively confined areas. Structure backfill specifications must be designed to insure construction of a durable, dense backfill. The following considerations (Table 6-1) must be addressed:

**Table 6-1**
**General Considerations for Select Structural Backfill**

| Consideration | Reason For |
|---|---|
| Lift Thickness | 6" to 8", so compaction possible with small equipment |
| Topsize | Less than ¾ of lift thickness |
| Gradation | Well graded for ease of compaction |
| Durability | Minimize breakdown of particles and settlement |
| Percent Fines | Minimize to prevent piping and allow rapid drainage |
| T99 Density Control | Small equipment cannot achieve AASHTO T180 densities |
| Compatibility | Particles should not move into voids of adjacent fill or drain material |

### 6.2.3 Estimate Settlement of the Approach Embankment Caused by Consolidation of the Subsoil

Many and varied procedures exist for computation of embankment settlement. Two methods will be presented herein; one each for cohesionless and cohesive soils. However, certain steps are common to either method, namely pressure distribution.

## 6.3 GENERAL PROCEDURE FOR APPROACH EMBANKMENT PRESSURE DISTRIBUTION

1. Plot soil profile including soil unit weights, SPT results (N), moisture contents and interpreted consolidation test values.

2. Draw overburden pressure ($P_o$) diagram with depth.

3. Plot total embankment pressure ($P_F$) on the $P_o$ diagram at ground surface level.

4. Distribute the total embankment pressure with depth using appropriate pressure coefficient charts. Figure 6-3 is a chart used for distribution of pressure beneath an approach embankment and end slope.

The fundamental principles to remember are that stresses from an embankment load spread out with depth in proportion to the embankment width and that the additional pressures on the soil decrease with depth.

### 6.3.1 Pressure Distribution Chart Use

Step 1. Determine the distance (b) from the centerline of the approach embankment to the midpoint of sideslope. Multiply the numerical value of "b" by the appropriate values shown on the right vertical axis of the chart to develop the depth at which the distributed pressures will be computed.

Step 2. Select the point (X) on the approach embankment where the settlement prediction is desired (normally at the intersection of the centerline of the embankment and the abutment). Measure the distance from this point X to the midpoint of the end slope. Return to the chart and scale that distance on the horizontal axis from the appropriate side of the midpoint of end slope line.

Step 3. Read vertically down from the plotted distance to the various curves corresponding to depth below surface. The "k" value on the left vertical axis should be read and recorded on a computation sheet with the corresponding depth.

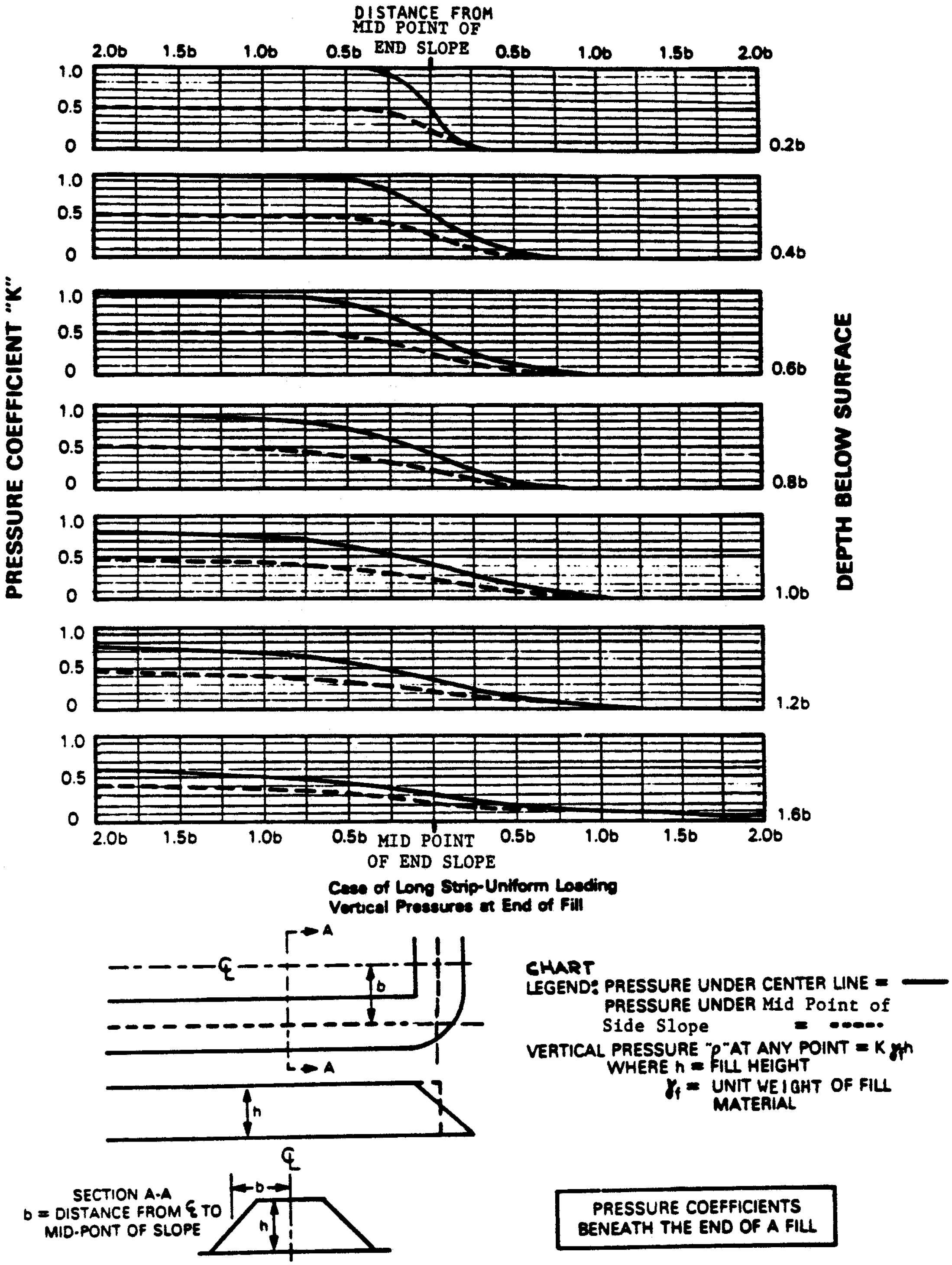



Figure 6-3: Pressure coefficients beneath the end of a fill

Step 4. Multiply each "k" value by the value of total embankment pressure to determine the amount of pressure ($\Delta P$) transmitted to each depth. The $\Delta P$ values should be added to the $P_o$ values at each depth to determine the final pressure ($P_F$). The $P_F$ values should then be plotted on the $P_o$ diagram and connected to form the $P_F$ line as follows:

Use Figure 6-3 charts (0.2B, 0.4B, etc) to find k values at depths from 0′ - 100′ ±. Multiply the k values times the embankment pressure $P_0$ to find $\Delta P$ at depths of 0.2B, 0.4B, etc. Add $\Delta P$ to $P_o$ at those depths and connect the points to produce a plot of $P_F$ as shown in Figure 6-4.

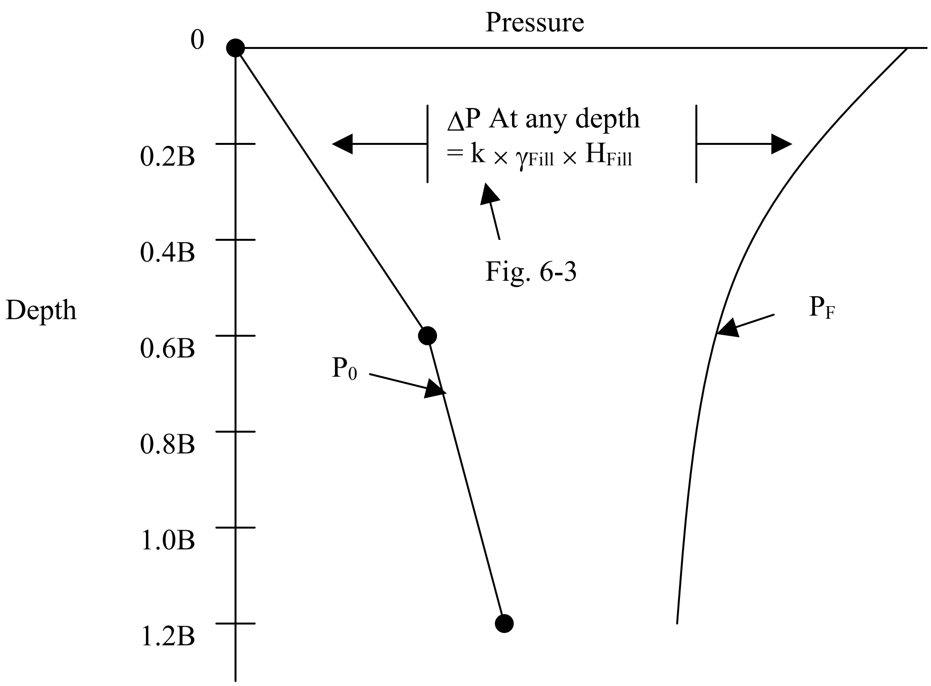
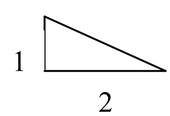


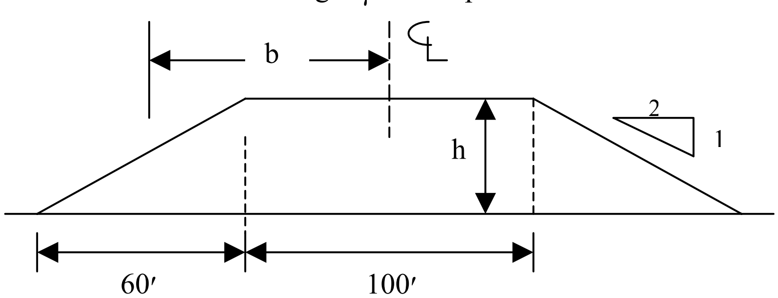

Figure 6-4: Plot of Pressure Increase with Depth Below an Embankment

**Example 6-1** - Use of Pressure Distribution Chart (Fig. 6-3)

Given: Fill height h = 30 ft.
End and side slopes (1V:2H)



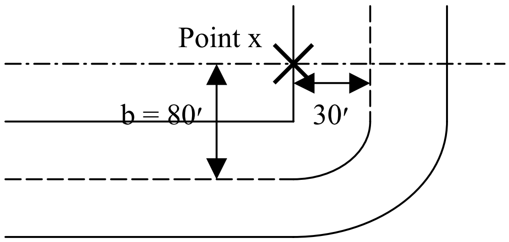

Embankment top width = 100 ft.
Fill unit weight $\gamma_F$ = 100 pcf




Distance from centerline (℄) to mid point of side slope $b = \dfrac{100}{2} + \dfrac{60}{2} = 80'$

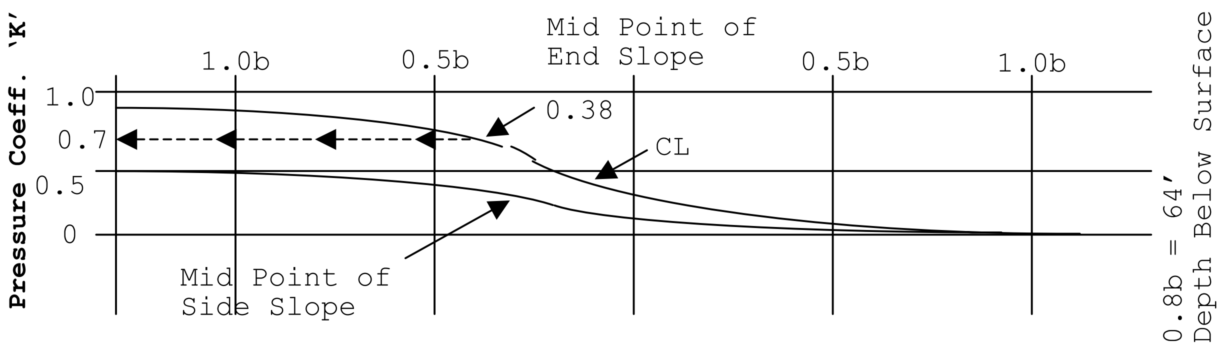
Find: The pressure increase ($\Delta P$) under the proposed abutment centroid (point x) at a depth of 0.8b (64ft).

below the base of the fill.




Solution: Distance from midpoint of end slope to point ‘x’ = 30′. ENTER PRESSURE DISTRIBUTION CHART FOR 0.8b depth at $\frac{30b}{80} = 0.38b$ distance from MIDPOINT OF END SLOPE.

**Pressure Distribution Chart**




From 0.8b (64′) depth chart read k = 0.7

$\therefore$ at 64′ depth $\Delta P = k\, \gamma_F\, h$ = (0.7)(100 pcf)(30 ft.)

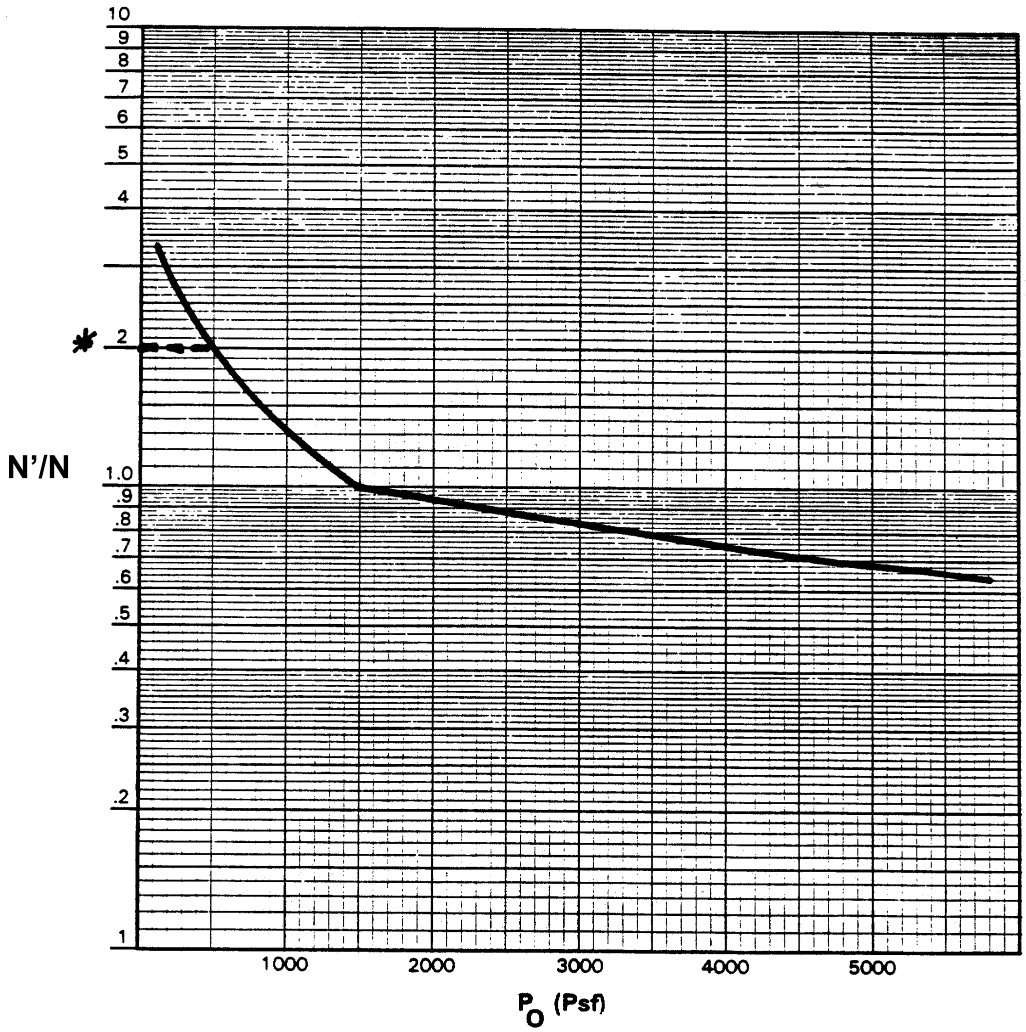
$\Delta P$ = 2100 psf

($\Delta P'_s$ at other depths found from other ″b″ charts)

## 6.4 SETTLEMENT COMPUTATION FOR COHESIONLESS SOILS

### 6.4.1 Correction of SPT Blow Counts

In recent years much attention has focused on the validity of field SPT blow counts (N). Numerous factors which influence SPT counts with increasing depth were investigated by field testing. The conclusion of that testing showed that physical factors such as increasing drill rod weight or rod flexibility had a minor overall effect on N counts. However, in non-cohesive soils, the increasing overburden pressure resulted in N values at increasing depths which indicated larger relative densities than actually existed. Conversely, at very shallow depths, where overburden pressures are low, N values indicated lower relative densities than actually existed. These overburden effects must be considered if correlations are to be made between N values and physical soil properties such as unit weight and friction angle. Therefore, N values from field operations must be corrected by the designer to reflect overburden pressure changes. Figure 6-5 should be used to obtain corrected SPT values, N′. In practice the maximum correction value should not exceed 2.




Where: N' = Corrected SPT Value Blow Count
N = SPT Value
$P_o$ = Existing Effective Vertical Overburden Pressure
* = Suggested Maximum Value

Reference: Based on 1967, Bazaraa, The Use of Standard Penetration Test for Estimating Settlement of Shallow Foundation on Sand

Figure 6-5: Correcting SPT (N) blow counts for overburden pressure, $P_o$

Step 1. Determine corrected SPT value (N′) from Figure 6-5.

Step 2. Determine Bearing Capacity Index (C′) by entering Figure 6-6 with N′ value and the visual description of the soil,

Step 3. Compute settlement in 10′ ± increments of depth from

$$\Delta H = H\left(\frac{1}{C'}\right)\text{Log}\frac{P_0 + \Delta P}{P_0} \tag{6-1}$$

Where:
- $\Delta H$ = Settlement (Feet)
- H = Thickness of soil layer considered (Feet)
- C′ = Bearing capacity index (Figure 6-6)
- $P_o$ = Existing effective overburden pressure (psf) at center of considered layer. For

shallow surface deposits, a minimum value of 200 psf must be used to prevent unrealistic computation of settlement.

$\Delta P$ = Distributed embankment pressure (psf) at center of considered layer

$P_F$ = Final pressure felt by foundation subsoil (psf)

Note: $P_F = P_o + \Delta P$

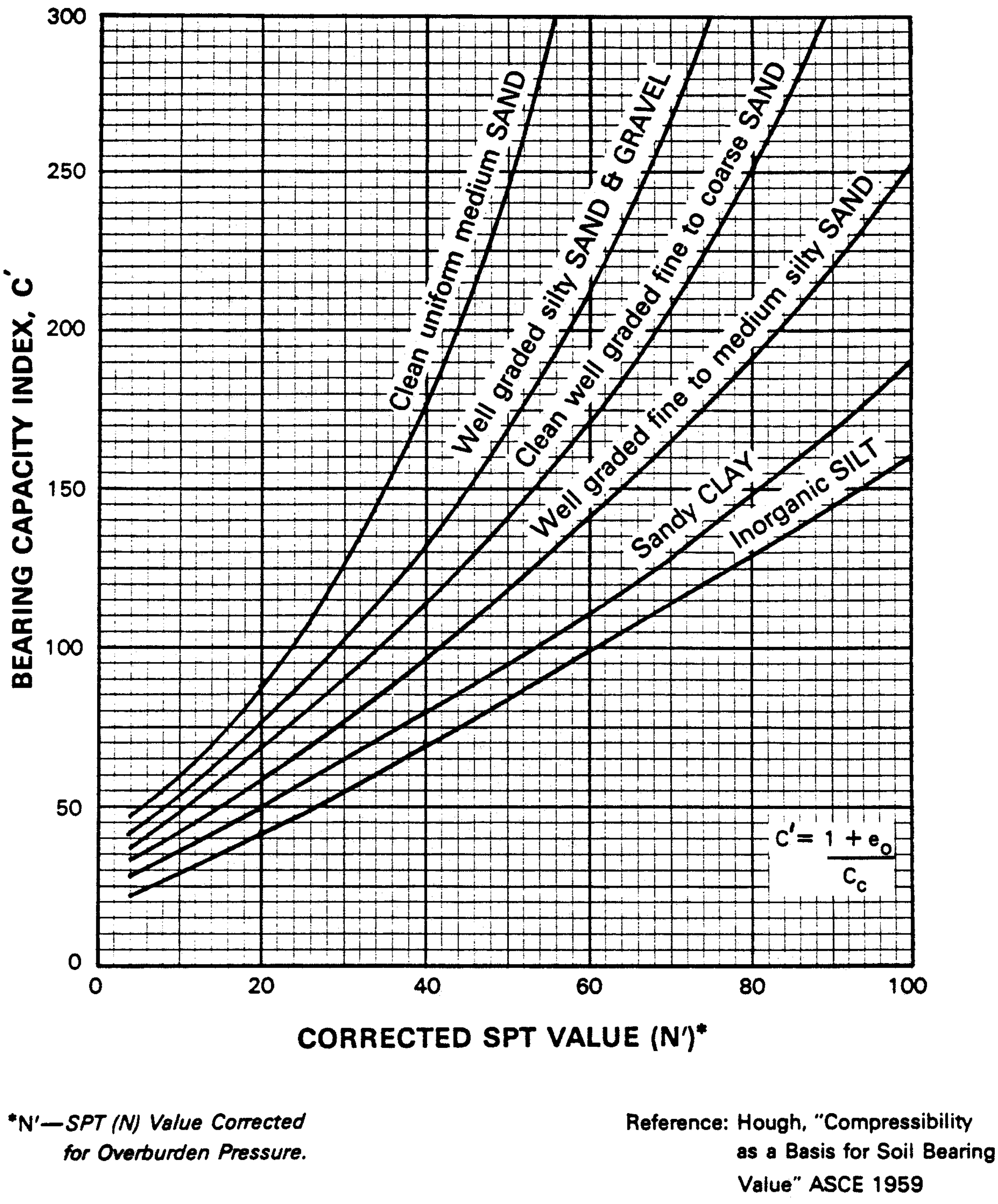


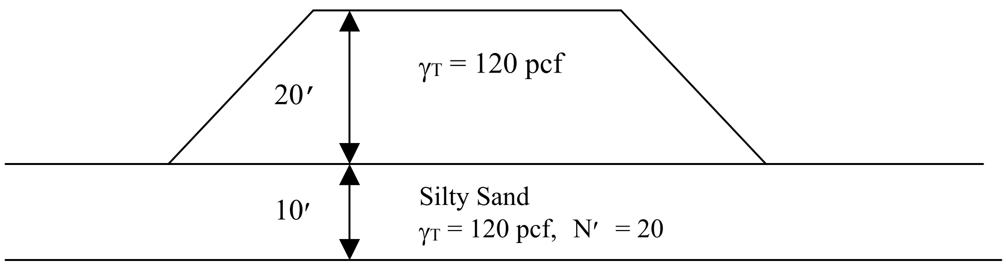

Figure 6-6: Bearing capacity index (C') values for granular soils

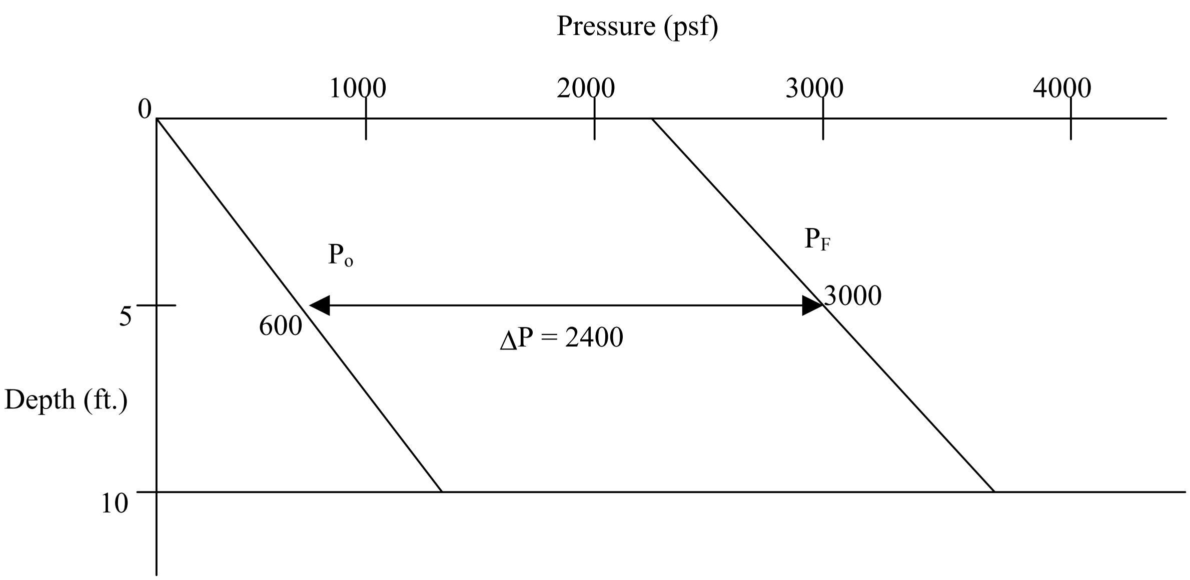
### 6.4.2 Time for Settlement

Time rate of settlement is not a concern for cohesionless soils. Cohesionless soils, being highly permeable, will settle instantaneously as load is applied. Embankment settlement amounts caused by consolidation of cohesionless soil deposits are frequently ignored because the settlement amounts are small in relation to cohesive deposits and the settlements occur as the embankment is constructed.

Time rate of consolidation settlement for cohesive soils is discussed in Section 6.7

**Example 6-2:** Determine The Settlement Of The Embankment Due To Consolidation Of The Silty Sand Layer Using The $P_O$ Diagram.




**Solution**

Find C′: Use N′ = 20 and Silty Sand Curve
In Figure 6-6
C′ = 58

Find Settlement

$$\Delta H = H \frac{1}{C'} \text{Log} \frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}1)$$

$$\Delta H = 10'\left(\frac{1}{58}\right)\text{Log}\ \frac{600\ \text{psf}\ + 2400\ \text{psf}}{600\ \text{psf}}$$

$$\Delta H = 0.12' = 1.44''$$

## 6.5 SETTLEMENT COMPUTATION FOR COHESIVE SOILS

1. Analyze consolidation test data to determine:

    a. Preconsolidation pressure ($P_c$)

    b. Initial void ratio ($e_o$) at $P_o$

    c. Compression and recompression indices ($C_c$ and $C_r$)

1. (ALT.) In the absence of consolidation test data, settlement may be approximated using Atterberg limit and moisture content data. This method is only recommended for use in final design if soils exist which are not suited for lab testing, i.e., surface muck deposits, etc.

    a. Soil may be assumed to be preconsolidated to pressures above typical embankment loadings if the liquidity index ([moisture content minus plastic limit] divided by plastic index) is less than 0.7.

    b. Initial void ratio, $e_o$, for saturated soils may be determined by multiplying the moisture content by the specific gravity and dividing by 100.

    c. $C_c$ and $C_r$ may be determined by dividing the moisture content by 100 and 1,000 respectively.

**Example 6-3:** Given moisture content 30, liquid limit 50, plastic limit 25, specific gravity 2.75. Find $e_0$, $C_c$, and $C_r$ and determine if the soil is preconsolidated.

**Solution:**

Liquidity index = $\frac{30-25}{50-25} = 0.2$ (preconsolidated, see "a" above)

$$e_o = \frac{(2.75)(30)}{100} = 0.825$$

$$C_r = \frac{30}{1000} = 0.03$$

$$C_c = \frac{30}{100} = 0.30$$

2. Compute settlement in 10′ ± increments of depth or at soil layer boundaries using:

$$\Delta H = H\left(\frac{C_c}{1+e_0}\right) Log \frac{P_0 + \Delta P}{P_0} \tag{6-2}$$

(For normally consolidated soils only, see later sections for preconsolidated soils)

Where:
- $\Delta H$ = Settlement (feet)
- $H$ = Thickness of soil layer considered (feet)
- $C_c$ = Compression index (from consolidation test)
- $C_r$ = Recompression index (from consolidation test)
- $e_o$ = Initial void ratio of soil
- $P_o$ = Existing effective overburden pressure (psf) at center of considered layer. For shallow surface deposits, a minimum value of 200 psf must be used to prevent unrealistic computation of settlement
- $\Delta P$ = Distributed embankment pressure (psf) at center of considered layer
- $P_F$ = Final pressure (psf) felt by foundation subsoil
- $P_F$ = $P_o + \Delta P$

Both the compression index, $C_c$, and the recompression index, $C_r$, may be used in settlement computations for preconsolidated clays. Only $C_c$ is used in settlement computations for normally consolidated clays.

### 6.5.1 Normally Consolidated Clay

For normally consolidated clays, the preconsolidation pressure $P_c$ is approximately equal to the existing overburden pressure $P_o$. This means that the soil has never in the past been loaded to a stress above that which presently exists in the ground, i.e., $P_o = P_c$.

$$\therefore \Delta H = H \frac{C_c}{1+e_0} Log \frac{P_F}{P_0} \tag{6-2a}$$

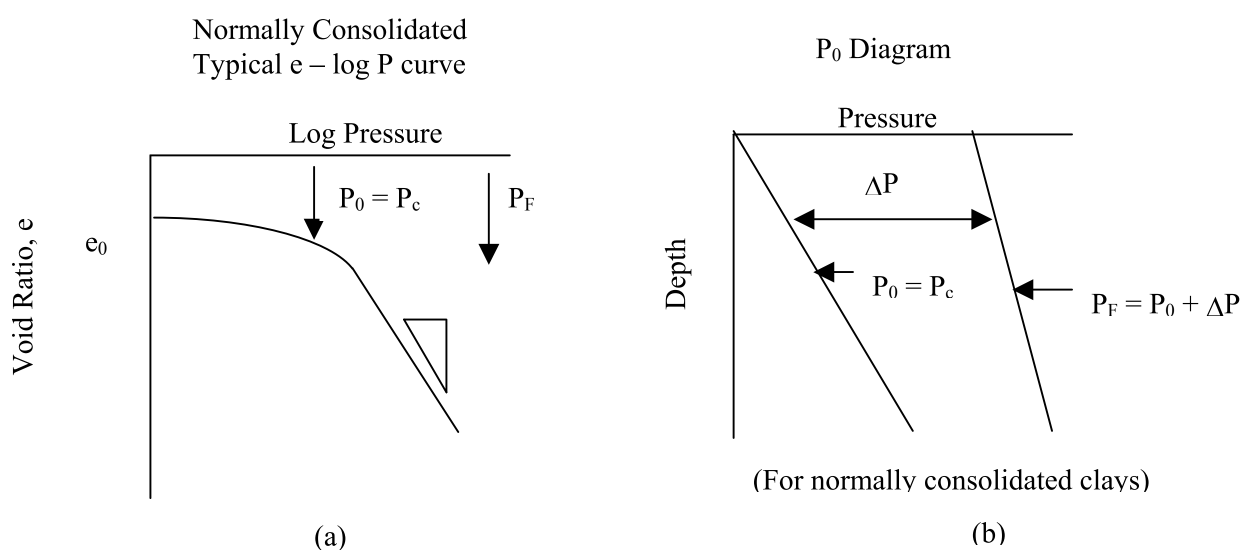



Figure 6-7: (a). Typical e-log P curve for Normally Consolidated Clay and, (b). Overburden Pressure ($P_0$) and Final Pressure Variation with Depth.

### 6.5.2 Preconsolidated Clay

The computation procedure for estimating settlement when preconsolidated clays exist in the soil profile is slightly more complicated. The computation is made much easier by use of the $P_o$ diagram (Figure 6-8). For preconsolidated clays, the preconsolidation pressure $P_c$ (determined from consolidation test) will be greater than the existing overburden pressure $P_o$ (Figure 6-8). This means that at some time in the past the clay has been subjected to a greater stress than now exists (due to weight of glaciers, weight of soil that has since eroded away, or due to desiccation).

For $P_F < P_c$

$$\Delta H = H \frac{C_r}{1+e_0} \text{Log} \frac{P_F}{P_0} \tag{6-2b}$$

For $P_F > P_c$

$$\Delta H = H \frac{C_r}{1+e_0} \text{Log} \frac{P_c}{P_0} + H \frac{C_c}{1+e_0} \text{Log} \frac{P_F}{P_c} \tag{6-2c}$$

These settlement analyses may be varied to judge the effects of excavation of unsuitable material, the placing of surcharges, or the substitution of lightweight fill materials.

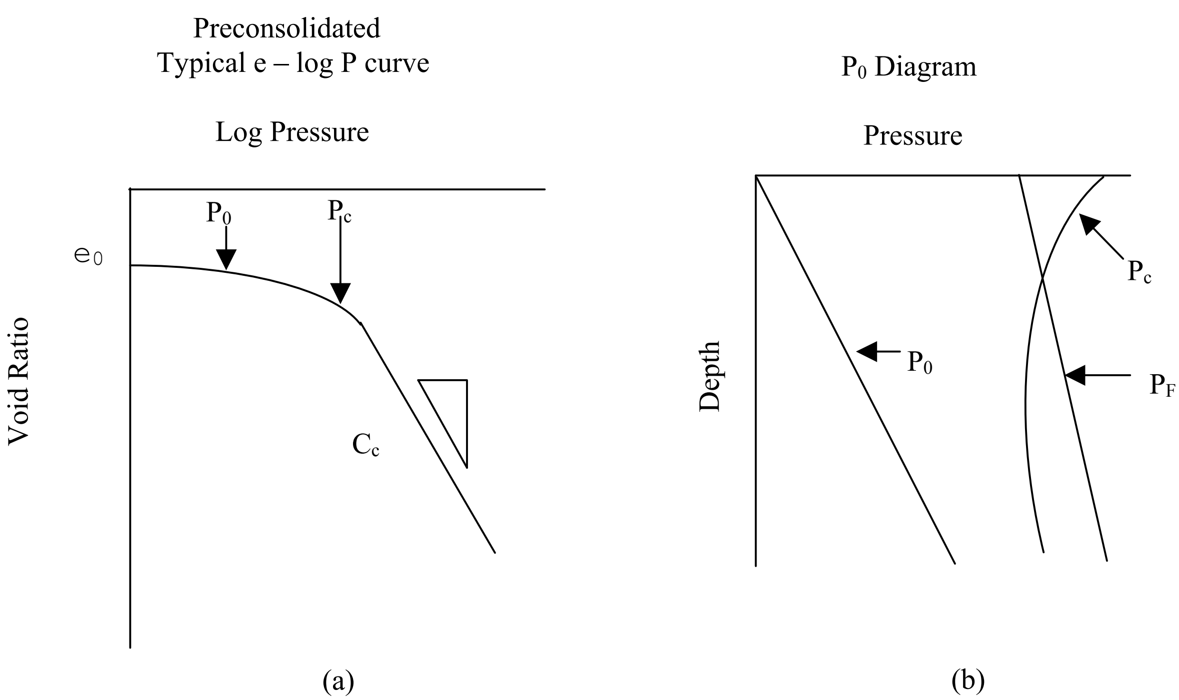



Figure 6-8: (a). Typical e-log P curve for Preconsolidation Clay and, (b). Variation of Overburden Pressure ($P_0$), Preconsolidation Pressure ($P_c$) and Final Pressure ($P_F$) with Depth.

## 6.6 ESTIMATING SECONDARY SETTLEMENT

Secondary settlement is of practical importance in soils containing organic material. Secondary settlement can occur for many years following construction. The "roller coaster" roadway that is typical for roads built across peat swamp deposits is sometimes due to long-term secondary settlements.

Secondary settlement can be estimated using the following relationship:

$$\Delta H_{sec} = C_\alpha \ H \ \text{Log} \frac{t_{sec}}{t_p} \tag{6-3}$$

Where:
- $\Delta H_{sec}$ = Secondary settlement
- $C_\alpha$ = Coefficient of secondary consolidation (determined from lab consolidation test)
- $H$ = Soil layer thickness
- $t_{sec}$ = Time over which secondary settlement is being estimated
- $t_p$ = Time for primary consolidation

Typically, the ratio $t_{sec}/t_p$ is taken as 10 when making the secondary settlement computation.

Approximate correlation of $C_\alpha$ versus natural water content is shown in Figure 6-9. The chart can be used to make secondary settlement estimate in the absence of consolidation test data.

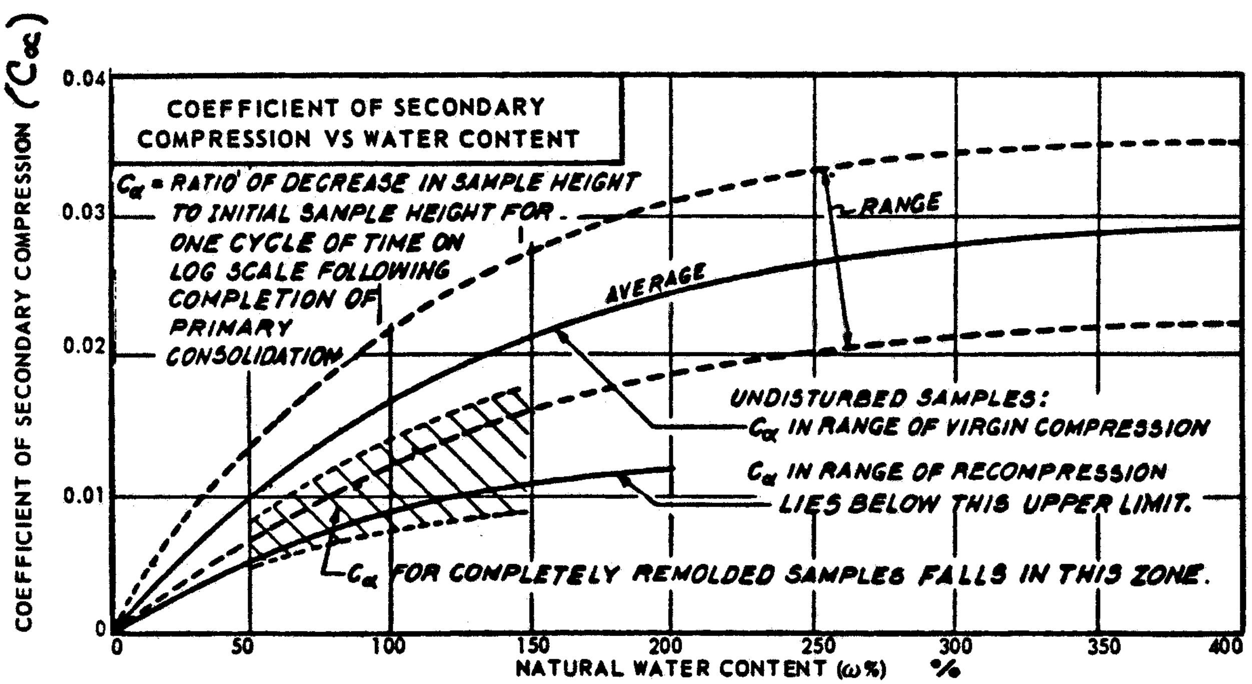



Figure 6-9: Correlation of $C_\alpha$ with Natural Water Contents

## 6.7 TIME RATE OF SETTLEMENT

In practice, settlement amount can be estimated with reasonable accuracy if the settlement estimate is based on properly conducted consolidation tests of quality undisturbed samples.

The time for the primary settlement to occur is more difficult to estimate. Commonly used formulas are based on consolidation of an assumed homogeneous soil deposit; a condition which seldom occurs in nature. Fortunately, however, the computed time estimate will usually be conservative, i.e., settlement in the field usually occurs more rapidly than the time estimated by theory. The reason is that most silt-clay deposits contain more permeable sand or silt lenses which provide lateral drainage and speed the settlement time. Careful attention should be paid during the field investigation to determine if such layers exist in the deposit. All extruded Shelby tube samples should be checked for presence of sand-gravel-silt lenses. Continuous Shelby tube samples taken in at least one boring can be very useful in determining if lenses are present.

The time rate of settlement can be estimated utilizing the coefficient of consolidation ($C_v$) obtained from consolidation testing. Since the time for 100 percent consolidation to occur is theoretically infinite, the time for 90 percent consolidation is usually considered the total time for primary settlement.

The time rate of settlement is based on a time factor and may be computed from

$$t = \frac{T{H_v}^2}{C_v} \tag{6-4}$$

Where:
- $t$ = time for settlement to occur (days)
- $T$ = theoretical time factor (dependent on percent consolidation as shown in Table 6-2)
- $H_v$ = maximum length of vertical drainage path in feet (single or double drainage)
- $C_v$ = coefficient of consolidation in feet squared per day ($C_v$ is obtained from the lab consolidation test using the time – compression curve for the test load increment midway between $P_o$ and $P_F$)

**Table 6-2**
**Time Factor (T)**

| Percent Primary Settlement | Time Factor (T) |
|---|---|
| 10 | 0.008 |
| 20 | 0.031 |
| 30 | 0.071 |
| 40 | 0.126 |
| 50 | 0.197 |
| 60 | 0.287 |
| 70 | 0.403 |
| 80 | 0.567 |
| 90 | 0.848 |

Settlement time can be computed, using the time factors for various percent primary settlements, to develop a predicted time-settlement curve for the field problem.

A typical time-settlement curve for a clay deposit under embankment loading is shown in Figure 6-10.

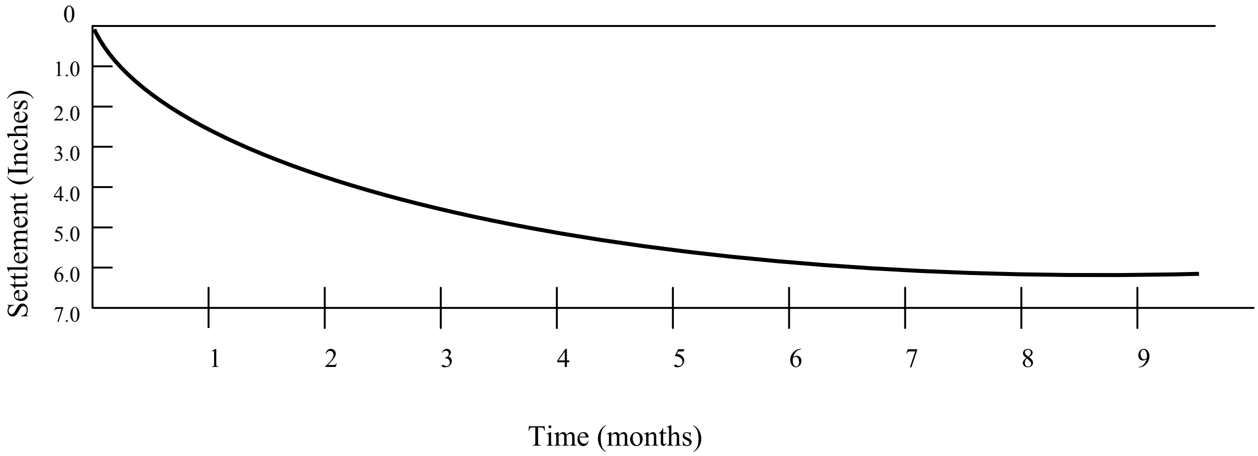
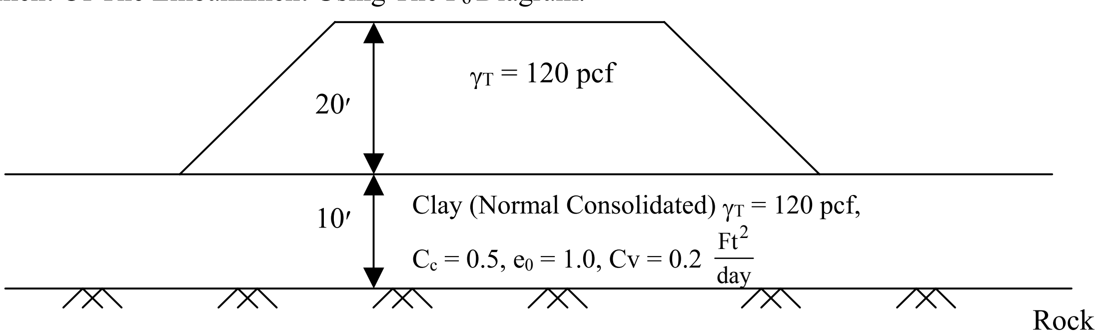


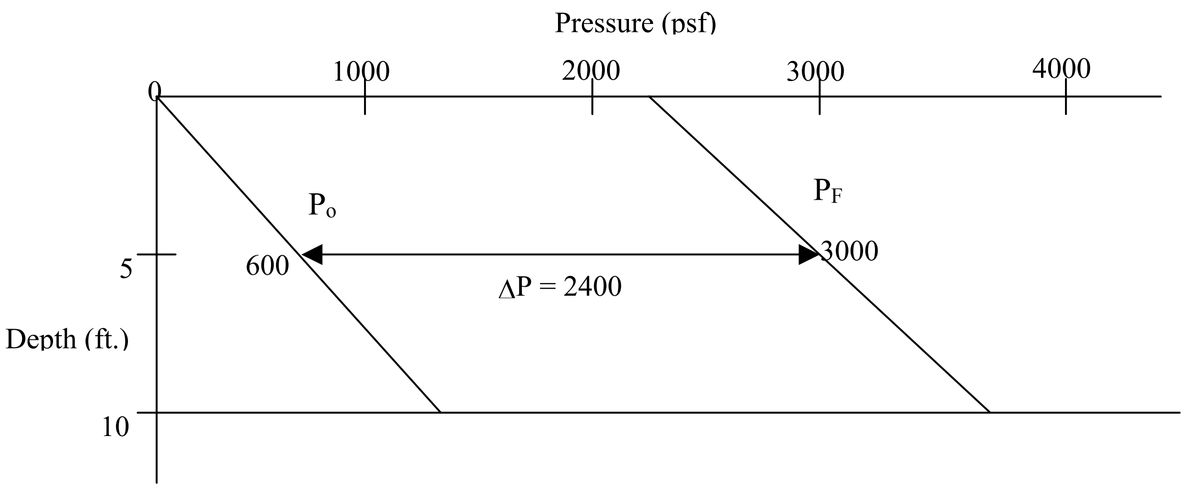

Figure 6-10: Typical Time-settlement Curve for Clay

Important factors to remember are both the time required for consolidation is proportional to the square of the longest distance required for water to drain from the deposit and the rate of settlement decreases as time increases. The maximum length of vertical drainage path, $H_v$, bears further explanation. This term should not be confused with the H term in the equation for settlement magnitude which is an arbitrarily selected value usually representing a portion of the total compressible layer thickness. The $H_v$ term is the maximum vertical distance that a water molecule must travel to escape from the compressible layer to a more permeable layer. In the case of a 20-foot thick clay layer bounded by a sand layer on top and a non-permeable rock strata on the bottom, the $H_v$ term would equal 20 feet. The water molecule must travel from the bottom of the layer to escape, i.e., single drainage. However, if the clay layer was bounded top and bottom by permeable sand deposits, the $H_v$ distance would be 10 feet. The water molecule in this case, needs only to travel from the center of the layer to either boundary to escape, i.e., double drainage.

Although horizontal drainage considerations are beyond the scope of this manual, the mechanism for determining the maximum horizontal path for escape of a water molecule is similar. The influence of horizontal drainage may be great if the width of the loaded area is small. For instance, during consolidation under a long, narrow embankment, a water molecule can escape by traveling a distance equal to one half the embankment width. However, for very wide embankments the beneficial effect of lateral drainage may be small as the time for lateral escape of a water molecule increases as the square of one-half the embankment width.

**Example 6-4:** Determine The Magnitude And The Time For 90% Consolidation For The Primary Settlement Of The Embankment Using The $P_o$ Diagram.






Solution:

Find Primary Settlement

$$\Delta H = H \frac{C_c}{1 + e_0} \, Log \frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}3)$$

$$= 10' \left( \frac{0.5}{1 + 1.0} \right) Log \frac{600 \, psf + 2400 \, psf}{600 \, psf}$$

$$\Delta \mathrm{H} = 1.75' = 21''$$

Find Time to 90% Consolidation:
Assume Single Vertical Drainage Due to Impervious Rock Layer.

$$\mathrm{t}_{90} = \frac{\mathrm{TH}^2}{\mathrm{C_v}} \qquad (6\text{-}4)$$

$$\mathrm{t}_{90} = \frac{(0.848)(10)^2}{0.2} = 424 \text{ days}$$

**6.8 DESIGN SOLUTIONS - SETTLEMENT PROBLEMS**

- Reducing Settlement Amount

Several of the methods for solving embankment stability problems can also be used to reduce the amount of settlement. These include:

1. Reduce grade line.
2. Excavate and replace soft soil.

3. Lightweight fill.
4. Stone columns

- Reducing Settlement Time

Often the major design consideration when faced with a settlement problem is the time for the settlement to occur. Low permeability clays and silt-clays can take a long time to consolidate (water squeezed out). The settlement time is generally what will get the chief engineer "excited," since this can affect construction schedules, increase project costs due to inflation, etc. Settlement time is also important to the maintenance forces of a highway agency. The life cycle cost of annual regrading and resurfacing of settling roadways is usually far greater than the cost of design treatments to eliminate settlement during initial construction.

The two most common methods used to accelerate settlement and reduce settlement time are:

1. Surcharge treatment.
2. Vertical drain treatment of subsoil.

### 6.8.1 Surcharge Treatment

An embankment surcharge is built up a predetermined amount, usually 1 to 10 feet, above final grade elevation and allowed to remain for a predetermined waiting period (typically 3 - 12 months). The actual dimensions of the surcharge and the waiting period will depend on the strength and drainage properties of the foundation soil as well as the initial height of the proposed embankment. The length of waiting period can be estimated using consolidation test data. The actual settlement occurring during embankment construction is then monitored with geotechnical instrumentation. When the settlement with surcharge equals the settlement originally estimated for the embankment the surcharge is removed, as illustrated in Figure 6-11.

The surcharge should not be left on after the desired settlement amount has occurred as additional settlement will occur. Note that the stability of a surcharged embankment must be checked to insure that an adequate safety factor exists to permit placement of the surcharge load.

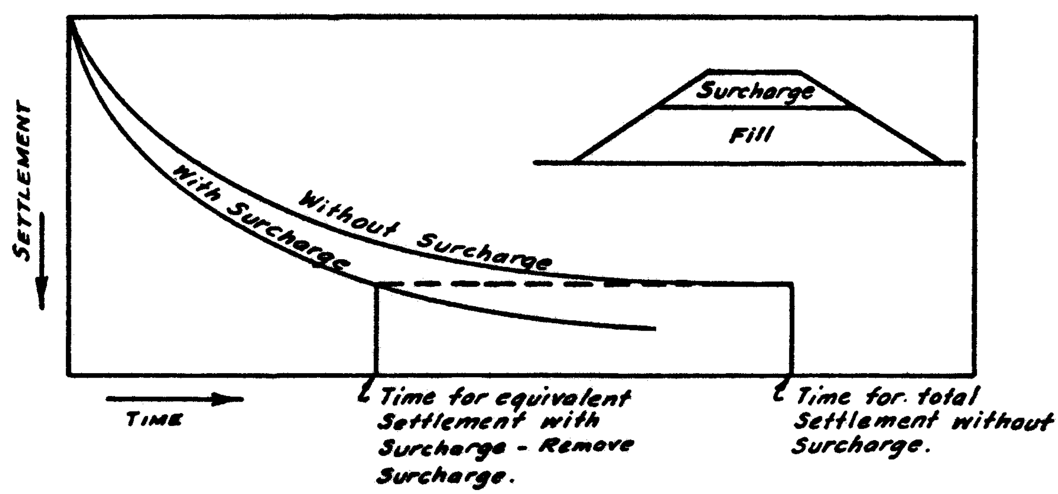



Figure 6-11: Determination of Surcharge Time Required to Achieve Desired Settlement

### 6.8.2 Vertical Drains

Some highly plastic clays of extremely low permeability can take many years for settlement to be completed. Surcharging alone may not be effective in reducing settlement time sufficiently. In such cases, vertical drains can be used to accelerate the settlement; either with or without the surcharge treatment. Although both sand and wick (prefabricated) types of vertical drains have been used in the past, predominately wicks have been used in recent years due to cost and environmental advantages. For either drain type, a permeable sand blanket, 2-3 feet thick, should be placed on the ground surface to permit movement of water away from the embankment area. The drains are installed prior to placement of the embankment as the pressure to drive the water up the vertical drains is caused by the embankment load. Surcharging should always be considered first, since vertical drains are generally more expensive. The reason vertical drains accelerate the settlement is that the drainage path the water must travel to escape from the impervious soil layer is shortened, as illustrated in Figure 6-12.

Recall that the settlement time is proportional to the square of the length of the drainage path, thus if the drainage path length can be cut in half, the time is reduced by a factor of four. The vertical drains and sand blanket must have high permeability to allow water squeezed out of the subsoil (due to the fill pressure) to travel up the drains and out through the blanket.

Wick drains are small prefabricated drains consisting of a plastic core which is wrapped by a piece of filter fabric. Wick drains are approximately 4 inches wide and about 1/4 inch thick and produced in rolls which can be fed into a mandrel. Wick drains are installed by pushing or vibrating a mandrel into the ground with the wick drain inside. When the bottom of the compressible soil is reached, the mandrel is withdrawn and the trimmed portion of the wick drain left in the ground. To minimize smear of the clay, the cross-sectional area of the mandrel is recommended to be limited to a maximum of about 10 square inches. Preholing of compact surface soil deposits may be required for mandrel installation. Use of wick drains in the United States began about 35 years ago. Wick drain projects will typically be 50 percent less costly than if sand drains were used. This is primarily due to much faster speed of installation and the environmental advantages of wick drains versus sand drains. Wick drains are now used almost exclusively in vertical drain applications.

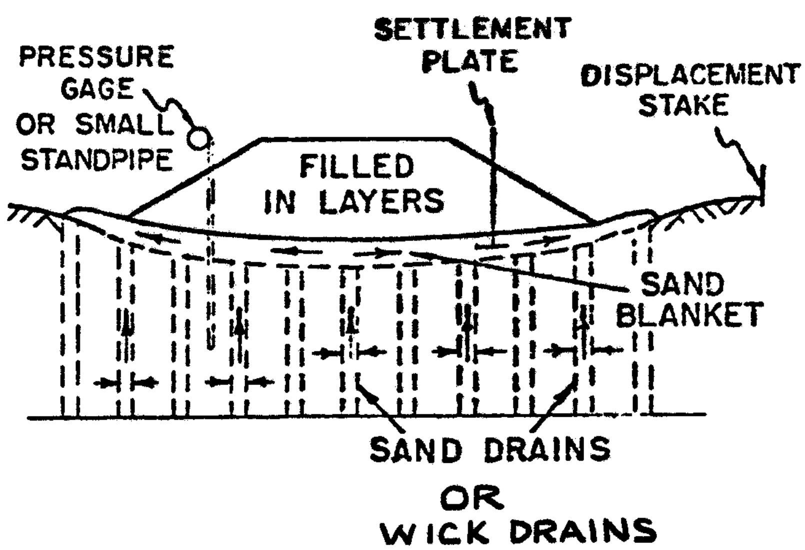



Figure 6-12: Use of Vertical Drains to Accelerate Settlement

## 6.9 PRACTICAL ASPECTS OF EMBANKMENT SETTLEMENT

Few engineers realize the influence of embankment placement on the subsoils. The total weight of an embankment has an impact on the type of foundation treatment that may be selected. For instance a relatively low height embankment of 10' may be effectively surcharged because the additional surcharge weight could be 30 to 40 percent of the proposed embankment weight. However, when the embankment height exceeds 50' the influence of a 5’ or 10’ trapezoid of soil on top of this heavy 50’ mass is small and probably not cost-effective. Conversely, as the embankment height (and, therefore, weight) increase, the use of a spread footing abutment becomes more attractive. A 30' high, 50' long approach embankment weighs about 15,000 tons compared to the insignificant weight of a total abutment loading which may equal 1,000 tons. Besides weight, the width of an embankment has an effect on total settlement. Wider embankments cause a pressure increase deeper into the subsoil. As might be expected, wide embankments will cause more settlement and will increase the time for consolidation to occur.

Also, the use of geotextiles or geocomposite drains can be an effective method of preventing the bump at the end of the bridge. It is suspected that high dynamic loads are routinely induced in the abutment backfill due to vehicle impact loads. Inadequate filter layers or non-durable drain aggregate can cause either piping of fines or accelerated pavement subsidence due to breakdown of aggregates. In geographic areas where select materials are not available, the use of geosynthetic reinforcement of the abutment backfill and approach area can reduce the bump at the end of the bridge.

Recent developments in microcomputer software now permit simple computer analysis of approach embankment settlement. Programs such as EMBANK permit the user to quickly compute settlements along abutments, piers buried in end slopes or pipes placed diagonally under approach fills.

## 6.10 APPLE FREEWAY DESIGN EXAMPLE – SETTLEMENT

In this chapter the Apple Freeway Example is used to illustrated the computation of settlement and time rate relationship. The options of surcharge and vertical drains are also examined.

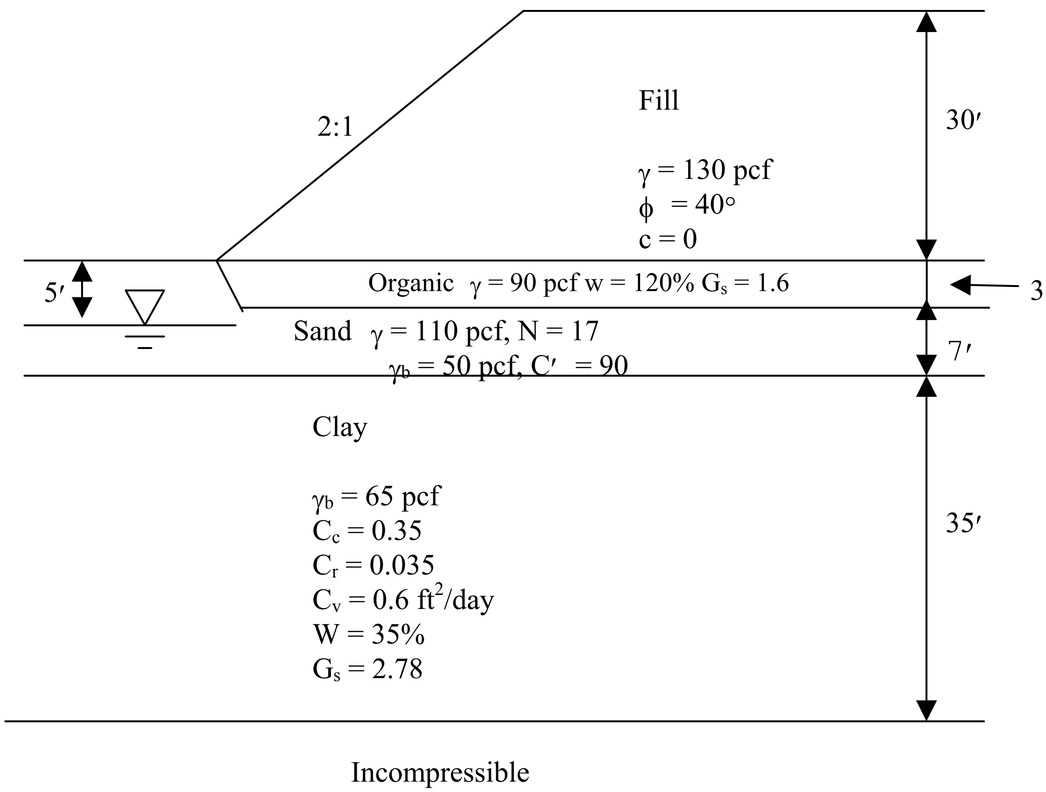
| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | Po Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| | Embankment Settlement |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

Design Soil Profile
Settlement
Time – Rate
Surcharge
Vertical Drains

Apple Freeway Design Example – Embankment Settlement
Exhibit A

**Given:** The Subsurface Profile and Soil Properties Shown Below, for the East Approach Embankment of the Apple Freeway Bridge.

**Required:** Compute the Magnitude and Time-rate of the Anticipated Settlement and Examine the Options of Surcharge and Using Vertical Drains (Including Cost Analysis)



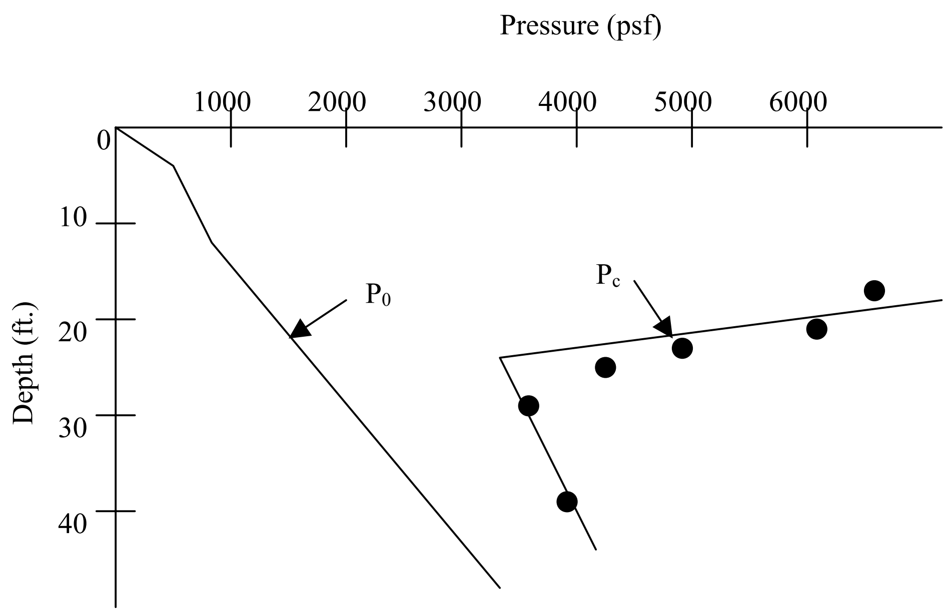

**Solution:**

**Step 1: Obtain Soil Consolidation Characteristics (from lab tests).**

**CONSOLIDATION TEST RESULTS**

| **Depth** | **Tube** | **$P_c$ (psf)** | **$C_c$** | **$C_r$** | **$C_v$ ($ft^2$/day)** |
|---|---|---|---|---|---|
| 11 | T3 | 6500 | 0.35 | 0.033 | 0.6 |
| 16 | T4 | 6000 | 0.32 | 0.031 | 0.4 |
| 21 | T5 | 4800 | 0.36 | 0.040 | 0.8 |
| 26 | T6 | 4200 | 0.34 | 0.035 | 0.6 |
| 31 | T7 | 3400 | 0.34 | 0.037 | 0.8 |
| 40 | T9 | 3800 | 0.35 | 0.032 | 0.4 |
| $e_0$ (average) = 0.97 | | | | | |

**Step 2:** **Plot Overburden Pressure and Preconsolidation Pressure Variation with Depth (below)**

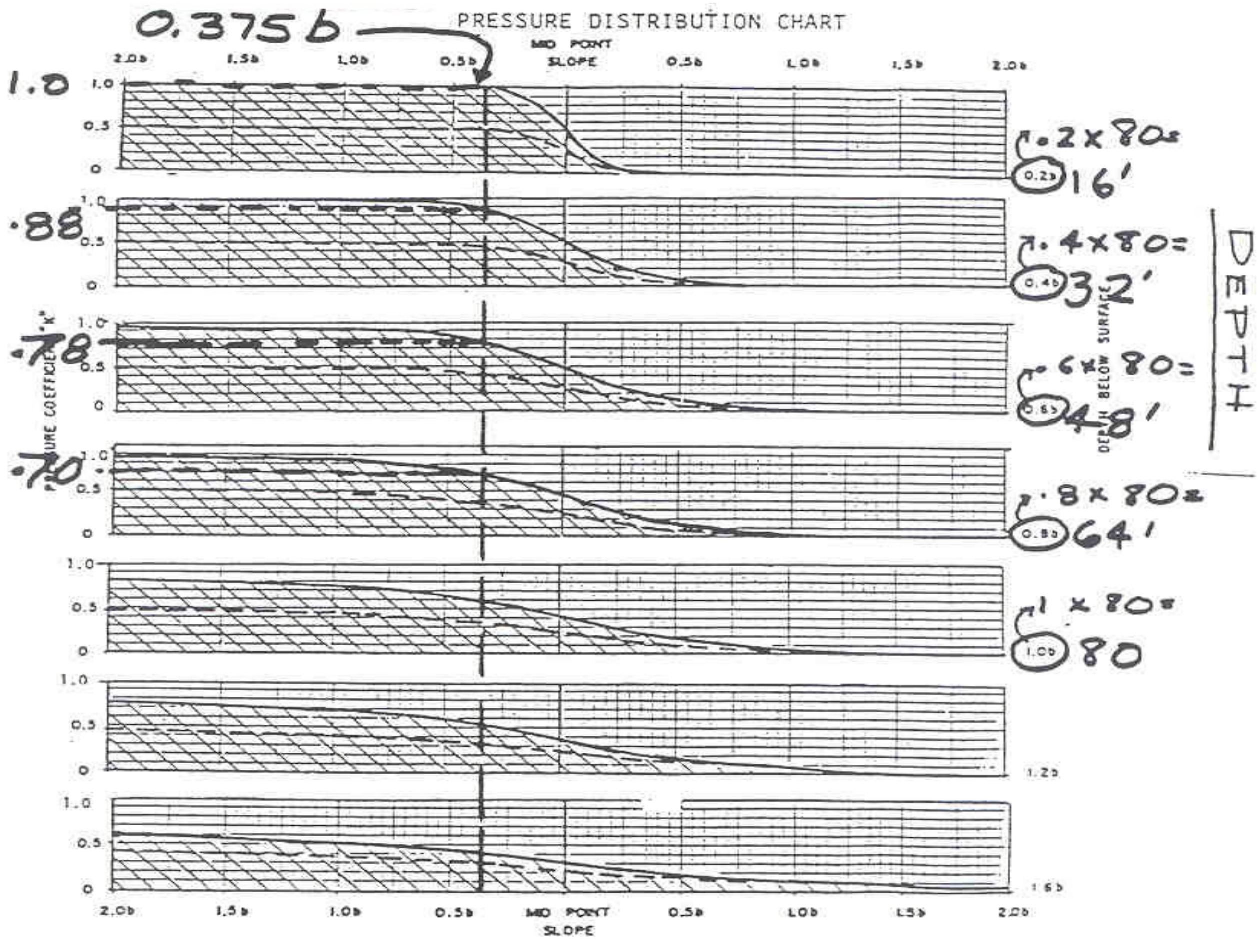


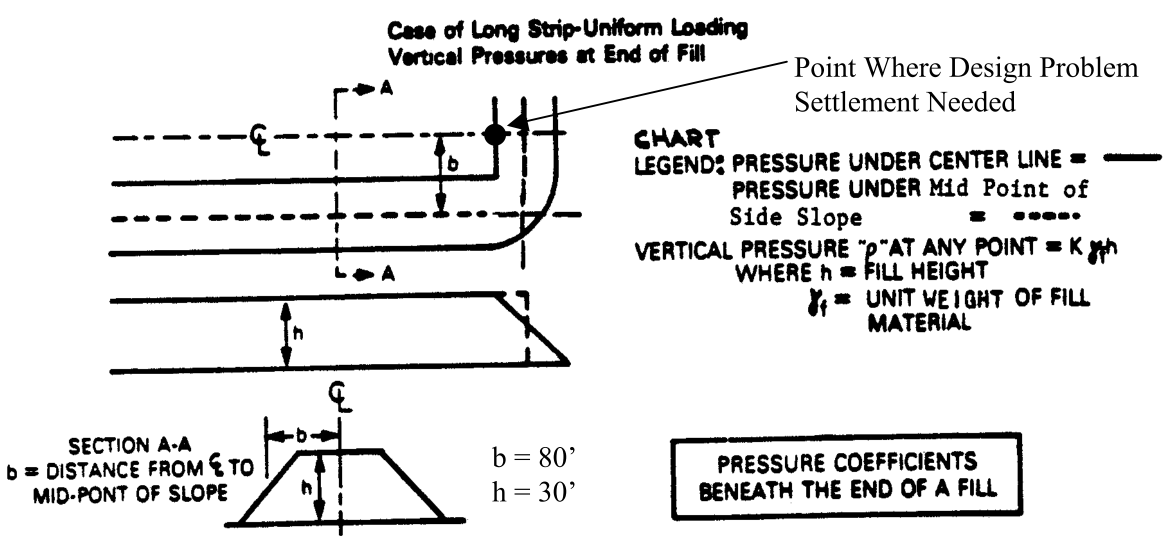

**Step 3:** **Determine Distribution of Final Embankment Pressure ($P_F$) with Depth:**

- Obtain embankment geometry (from Plan and Section).
- Embankment top width = 100′
- Side & end slopes 1V on 2H
- Top of end slope 60′ from toe
- Embankment height = 30′
- Embankment load (at center) = $H_{emb} \times \gamma_{emb}$
  $= 30' \times 130$ pcf $= 3900$ psf

- Abutment center located 30′ from midpoint of end slope $\rightarrow \frac{30}{80}b = 0.375b$
- Go to pressure distribution chart with b = 80′ $\left(\frac{100}{2} + \frac{60}{2}\right)$ and a distance from midpoint of end slope of 0.375b and obtain "K"
- Compute Pressure Change $\Delta P = K \times$ embankment load.

| Depth | "K" | $\Delta P$ = "K" × 3900 Distributed pressure (psf) |
|---|---|---|
| 16′ | 1.00 | 3900 |
| 32′ | 0.88 | 3432 |
| 48′ | 0.78 | 3042 |
| 64′ | 0.70 | 2730 |


0.375b
PRESSURE DISTRIBUTION CHART
MID POINT SLOPE
2.0b 1.5b 1.0b 0.5b 0.5b 1.0b 1.5b 2.0b
1.0
.88
.78
.70
PRESSURE COEFFICIENT "k"
.2 x 80 = 16'
.4 x 80 = 32'
.6 x 80 = 48'
.8 x 80 = 64'
1 x 80 = 80
0.2b
0.4b
0.6b
0.8b
1.0b
1.2b
1.6b
DEPTH BELOW SURFACE
DEPTH

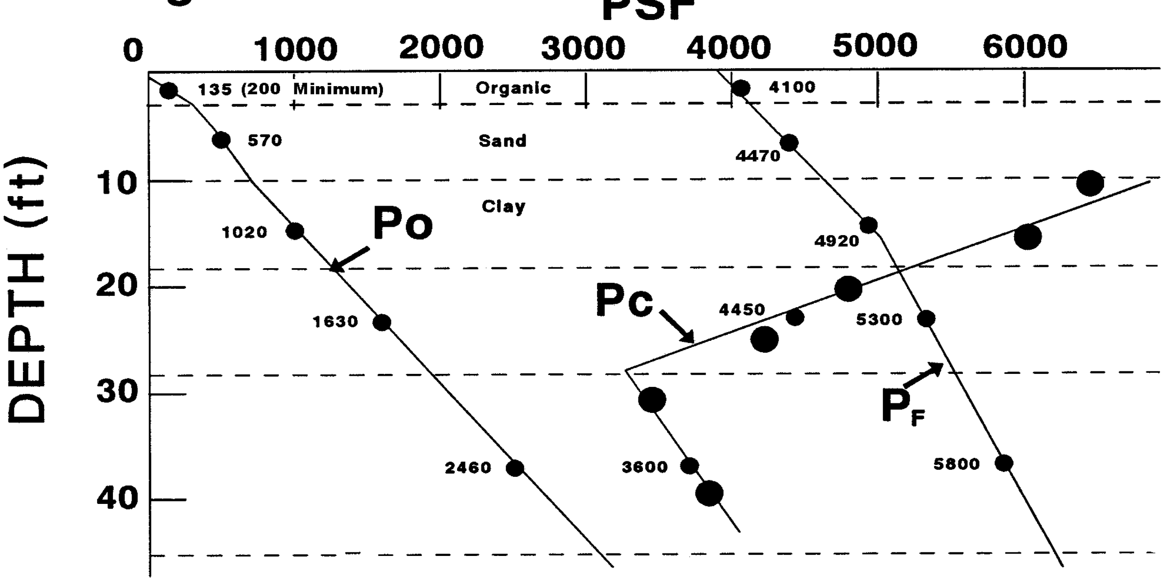

Case of Long Strip-Uniform Loading
Vertical Pressures at End of Fill
Point Where Design Problem Settlement Needed
A
b
h
CHART
LEGEND: PRESSURE UNDER CENTER LINE =
PRESSURE UNDER Mid Point of Side Slope =
VERTICAL PRESSURE "p" AT ANY POINT = K $\gamma_f$h
WHERE h = FILL HEIGHT
$\gamma_f$ = UNIT WEIGHT OF FILL MATERIAL
SECTION A-A
b = DISTANCE FROM ℄ TO MID-PONT OF SLOPE
b = 80'
h = 30'
PRESSURE COEFFICIENTS BENEATH THE END OF A FILL

**Step 4: Plot $P_0$, $P_c$ and $P_F$ with depth**

$P_F = P_0 + \Delta P$

Plot $P_F$ on $P_0$ diagram

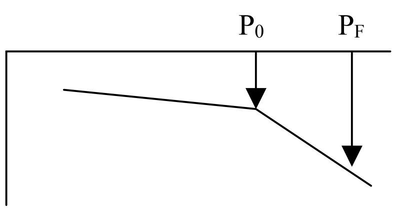
## Po Diagram




In settlement analysis, use pressures measured at center of layer or partial layer. Thick layers should be subdivided (ie. if layer is 20′ thick. compute settlement in 10′ increments) unless the slope of $P_o$, $P_c$, or $P_F$ are slowly converging straight lines. Dashed lines in above diagram show selected increments for analysis.

**Step 5: Compute settlement in each layer (or partial layer).**

- Layer 1 – Organic (0′ to 3′)




$$\Delta H = H \frac{C_c}{1+e_0} \text{Log} \frac{P_F}{P_0}$$

$H = 3' - 0' = 3'$

$$C_c = \frac{w}{100} = \frac{120}{100} = 1.2$$

$$e_0 = \frac{w \times G_s}{\%Sat.} = \frac{120 \times 1.6}{100} = 1.9$$

$$\Delta H = 3\left(\frac{1.2}{1+1.9}\right)\text{Log}\frac{4100}{200*} \qquad \text{* Remember } (P_0 \geq 200\text{psf})$$

$$\Delta H = 1.63' = 19.54''$$

- Layer 2 – Sand (3′ to 10′)

$$\Delta H = H\frac{1}{C'}\text{Log}\frac{P_F}{P_0}$$

H = 10′ - 3′ = 7′

To find C′ use N = 17 (BAF – 3)

$$\frac{N'}{N} = 2 \ @ \ P_0 = 500\,\text{psf}\,(\text{Figure } 6-5)$$

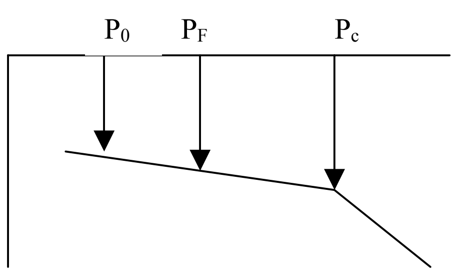
N’ = 34

C′ = 90 (Figure 6-6 between silty sand & fine to coarse sand)

$$\Delta H = (7)\left(\frac{1}{90}\right)\text{Log}\frac{4470}{570}$$

$$\Delta H = 0.069' = 0.83''$$

- Layer 3 – Clay (10′ to 18′)




$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_0}$$

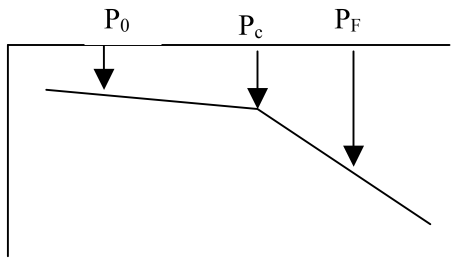
H = 18′ - 10′ = 8′

From Consol. Test data:
$C_r$ (avg.) = 0.035
$e_0$ (avg.) = 0.97

$$\Delta H = (8)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{4920}{1020}$$

$$\Delta H = 0.097' = 1.17''$$

- Layer 3 – Clay (18′ to 28′)




28′ chosen as $P_c$ slope changes

Compute $\Delta H$ separately for $P_0 > P_c$ and $P_c > P_F$

$$\Delta H = H\frac{C_r}{1+e_0}\text{Log}\frac{P_c}{P_0} + H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_c}$$

H = 28′ - 18′ = 10′

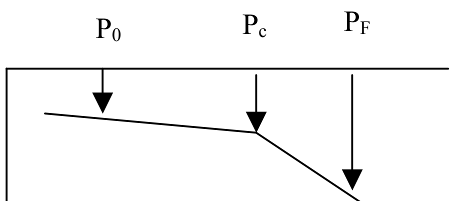
From Consol. Test data:

$C_r$ (avg.) = 0.035
$C_c$ (avg.) = 0.35
$e_0$ (avg.) = 0.97

$$\Delta H = (10)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{4450}{1630} + 10\left(\frac{0.35}{1+0.97}\right)\log\frac{5300}{4450}$$

$$\Delta H = 0.077' + 0.135' = 0.93'' + 1.62'' = 2.55''$$

- Layer 3 – Clay (28′ to 45′)




$$\Delta H = H\frac{C_r}{1+e_0}\text{Log}\frac{P_c}{P_0} + H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_c}$$

H = 45′ - 28′ = 17′

From Consol. Test data:

$C_r$ (avg.) = 0.035
$C_c$ (avg.) = 0.35

$e_0$ (avg.) = 0.97

$$\Delta H = (17)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{3600}{2460} + (17)\left(\frac{0.35}{1+0.97}\right)\text{Log}\frac{5800}{3600}$$

$$\Delta H = 0.050' + 0.63' = 0.60'' + 7.51'' = 8.11''$$

| Total Settlement | |
|---|---|
| Layer 1 – Organic (0′ to 3′) | 19.54″ |
| Layer 2 – Sand (3′ to 10′) | 0.83″ |
| Layer 3 – Clay (10′ to 18′) | 1.17″ |
| Clay (18′ to 28′) | 2.55″ |
| Clay (28′ to 45′) | 8.11″ |
| $\Delta H_{Total}$ | 32.20″ |

Assume organic layer is excavated and compacted select material placed.

$\Delta H$ of 19.54″ in organic layer will be eliminated after excavation of organic layer: $\Delta H_{Total}$ = 12.66″.

**Step 6: Compute Time for Settlement to Occur**

- Layer 1 – Select backfill material no settlement expected.
- Layer 2 – 0.83″ settlement occurs immediately in sand.
- Layer 3 – $\Delta H$ = 12.66″ - 0.83″ = 11.8″

Time t computed from: $t = \dfrac{T\,H_V^2}{C_v}$

$H_v$ = Drainage path
$C_v$ = 0.6 ft$^2$/day
T = From time factor chart

$H_v$ = ½ thickness of clay layer since permeable layers exist above and below

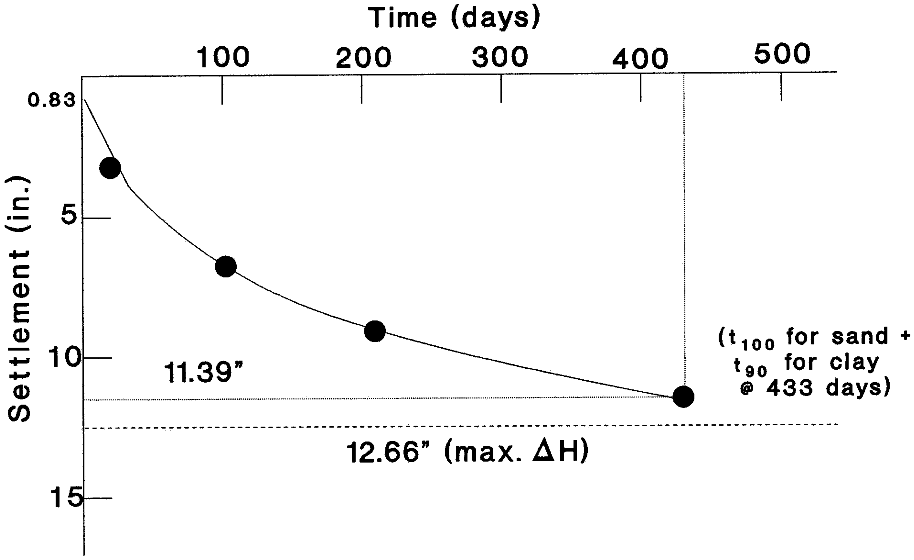
$$H_v = \frac{35'}{2} = 17.5'$$

| **% Consol. Layer 3** | **Layer 3 $\Delta H$ (in.)** | **T** | **$\frac{H_v^2}{C_v}$** | **t (days)** |
|---|---|---|---|---|
| 20 | 2.4 | 0.031 | 510.4 | 16 |
| 50 | 5.9 | 0.197 | | 101 |
| 70 | 8.3 | 0.403 | | 206 |
| 90 | 10.6 | 0.848 | | 433 |

The time-settlement plot can now be constructed for all soil layers
$\Delta H_{Total}$ = 12.66″

Remember to include 0.83″ sand settlement which occurs immediately as load is applied.

**Step 7: Plot Time – Settlement Curve**




The designer must insure that 90% consolidation is achieved before construction of the abutment foundation. Choices of treatment are:

1. A 433 day (14 mo.) waiting period
2. Surcharge
3. Vertical drains

**Examine Surcharge Option**

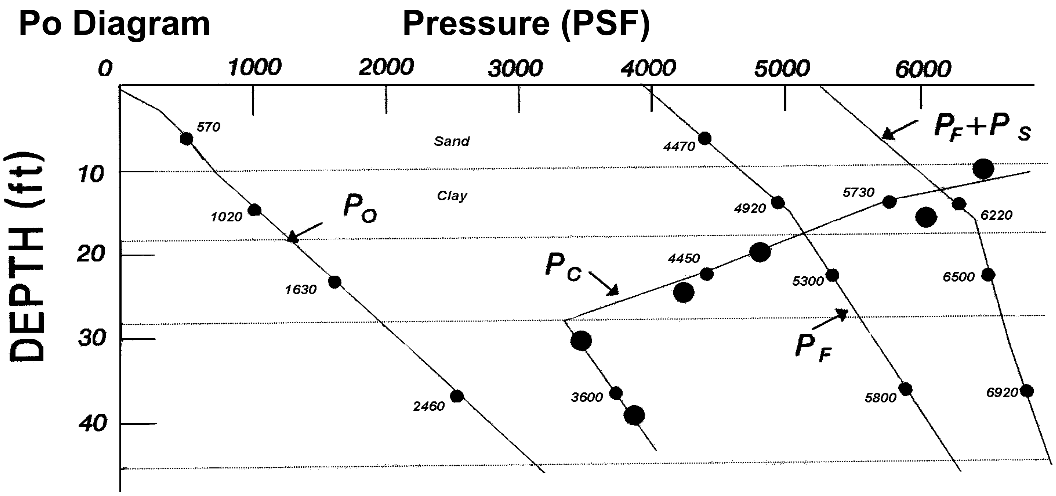
Assume:

- 10′ high compacted surcharged ($\gamma$ = 130 pcf), $\Delta$P of emb. ($P_F$) + Surch. ($P_s$) = 5,200 psf.
- Pressure distribution ″K″ value unchanged, additional consolidation of sand is negligible.
- $e_0$ remains 0.97 although the actual value is less due to compression under the previous load.

**Step 1: Obtain pressure increase with depth (use previous "K" value)**

| Depth Below OGS | K | K (5200 psf) |
|---|---|---|
| 0.2b = 16′ | 1.00 | 5200 |
| 0.4b = 32′ | 0.88 | 4580 |
| 0.6b = 48′ | 0.78 | 4060 |

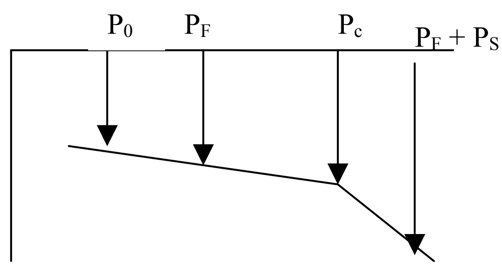
**Step 2: Plot Pressure Diagram**




**Step 3: Compute Settlement in layer 3 (only layer with additional settlement).**

Settlement

- Layer 3 – Clay (10′ to 18′)




$$\Delta H = H\frac{C_r}{1+e_0}\text{Log}\frac{P_c}{P_F}$$

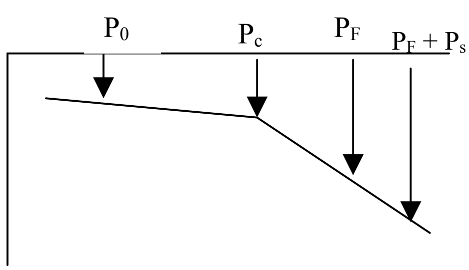
$$\Delta H = (8)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{5730}{4920} = 0.01' = 0.12''$$

$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F+P_s}{P_c}$$

$$\Delta H = (8)\left(\frac{0.35}{1+0.97}\right)\text{Log}\frac{6220}{5730} = 0.051' = 0.61''$$

$$\Delta H = 0.12'' + 0.61'' = 0.73''$$

- Layer 3 – Clay (18′ to 28′)

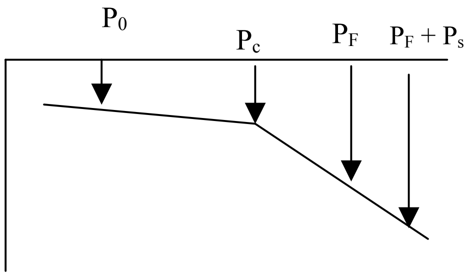



$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F+P_s}{P_F}$$

$$\Delta H = (10)\left(\frac{0.35}{1+0.97}\right) Log \frac{6500}{5300}$$

$$\Delta H = 0.157' = 1.89''$$

- Layer 3 – Clay (28′ to 45′)




$$\Delta H = H \frac{C_c}{1+e_0} Log \frac{P_F + P_s}{P_F}$$

$$\Delta H = (17)\left(\frac{0.35}{1+0.97}\right) Log \frac{6920}{5800}$$

$$\Delta H = 0.394' = 4.72''$$

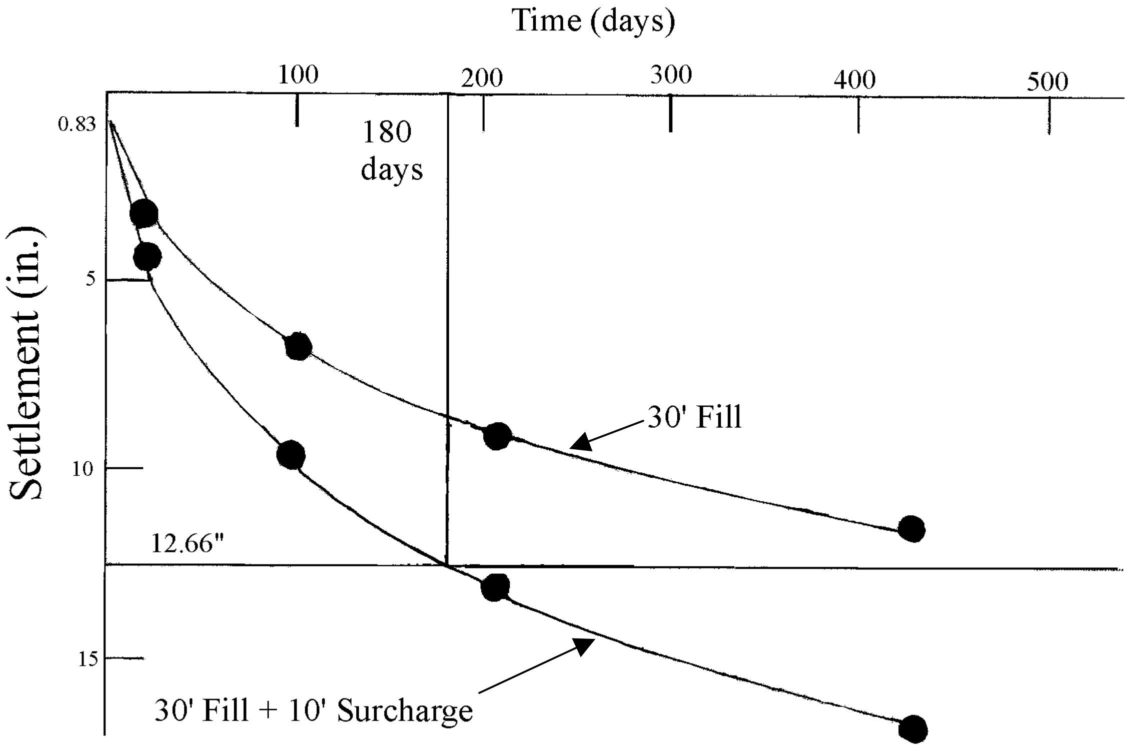
| Layer | Embank. | Surch. | Combined |
|---|---|---|---|
| 10′ to 18′ | 1.17″ | 0.73″ | 1.90″ |
| 18′ to 28′ | 2.55″ | 1.89″ | 4.44″ |
| 28′ to 45′ | 8.11″ | 4.72″ | 12.83″ |
| | | Total ΔH (clay Layer) = | 19.17″ |

**Step 4: Obtain Time-Settlement Relationship:** $\mathbf{t = \frac{T\,H_v^{\,2}}{C_v}}$ .

| %U | ΔH Clay (inches) | T | $\frac{H_v^{\,2}}{C_v}$ | T (days) |
|---|---|---|---|---|
| 20 | 3.8″ | 0.031 | 510.4 | 16 |
| 50 | 9.6″ | 0.197 | | 101 |
| 70 | 13.4″ | 0.403 | | 206 |
| 90 | 17.3″ | 0.848 | | 433 |

**Step 5: Plot time-settlement curve.**

**Time – Settlement Plot (includes ΔH Sand = 0.83″)**



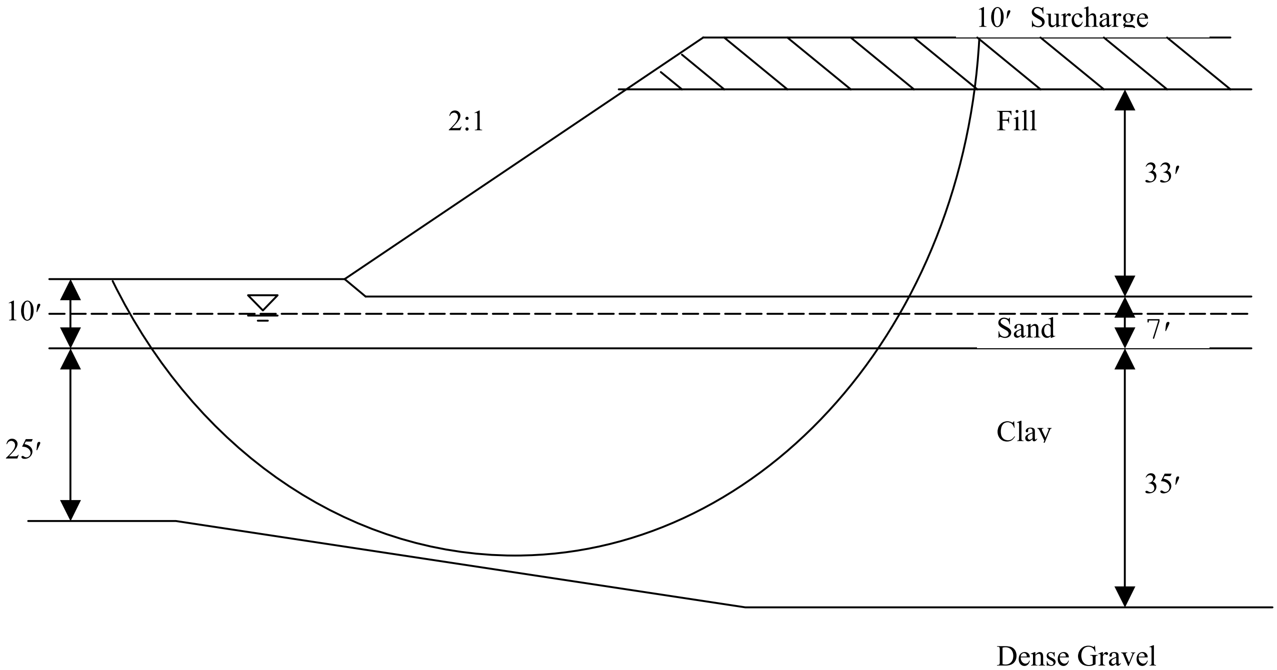

**Step 6: Determine time of waiting period with surcharge to obtain equivalent settlement to that of proposed embankment.**

Enter time – settlement plot for 30′ fill with 10′ surcharge with 12.66″ (settlement expected for 30′ fill). Extend line across to 30′ fill + 10′ surcharge curve and read waiting period time in days from time axis, ie. 180 days or 6 months.

**Step 7: Recommended instrumentation for monitoring settlement:**

| Instrument | Station | Depth Below Ground |
|---|---|---|
| Settlement plate | 90 + 00 | At ground surface |
| Settlement plate | 93 + 50 | At ground surface |
| Settlement plate | 96 + 50 | At ground surface |
| Piezometers | 93 + 50 | 20′, 28′, 36′ |
| Piezometers | 96 + 50 | 20′, 28′, 36′ |

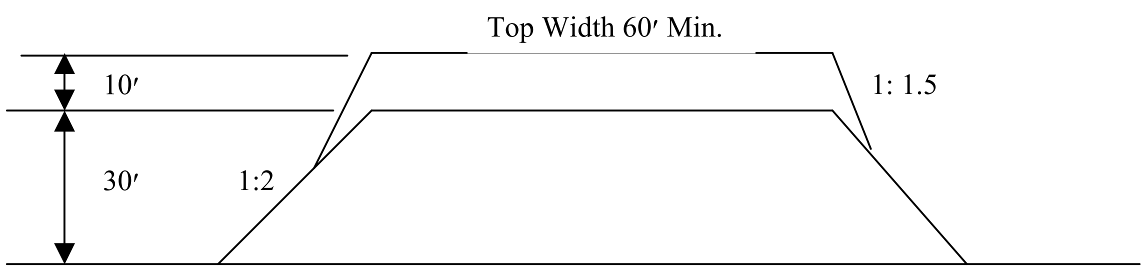
Step 8: **Recheck stability of 30′ fill with 10′ surcharge**




Safety Factor (w/surcharge) = 1.33 (1.63 w/o surcharge)

As safety factor higher than 1.30 (which is minimum recommended for bridge approach stability is O.K)

**Step 9: Prepare cost estimate for surcharge**




500 linear feet behind top of end slope to be surcharged at each approach.

Total 1000′

Surcharge quantity (avg. width = 80′ including side slopes)

$$80 \times 10 \times \frac{1000}{27} = 29{,}628 \text{ C.Y.}$$

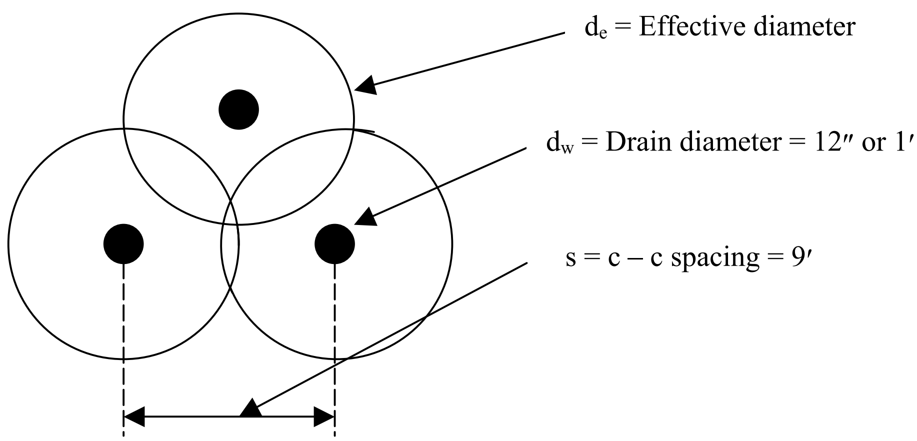
Cost to place and remove surcharge assumed at \$4.00 / C.Y.

Total cost = 29,628 C.Y. $\times$ \$4.00 / C.Y. = \$120,000.00

**Examine option of Sand Drains – No Surcharge**

**Step 1: Choose reasonable spacing of sand drains ie. use 12″ diameter on 9′ center to center triangular spacing.**

$C_v = 0.6\ ft^2/day$, assume $C_H = 0.6\ ft^2/day$ also.




**Step 2: Compute time-settlement relationship**

For triangular spacing: $d_e = 1.05 \times s$

$= 1.05 \times 9' = 9.5'$

$\eta = d_e/d_w = 9.5' / 1' = 9.5'$ (Say 10′)

Use time factor curves for radial drainage such in FHWA-RD-86-168; Figure 4.

Sand Drains – No surcharge

| $U_R$ % Consolidation | $T_R$ = (Radial Time Factor) |
|---|---|
| 20 | 0.045 |
| 50 | 0.140 |
| 70 | 0.230 |
| 90 | 0.450 |

- Check $t_{90}$ for radial drainage to see if assumed 9′ spacing is effective

$t_{90} = T_R\, d_e^2/C_H$
$= (0.45)(9.5)^2/(0.6) = 68$ days
OK ($t_{90}$ w/o drains = 433 days)

- Check time – settlement for combined vertical and radial drainage.

  $U_C$ = % consolidation for combined drainage = 100% - [(100% - $U_R$)(100% -$U_V$)]

  Assume time (in days) to compute $U_C$

- Check for t = 30 days

  $T_R = t\, C_H/d_e^2 = (30)(0.6)/(9.5)^2 = 0.20$

  $U_R$ = 64% (from FHWA-RD-86-168; Figure 4, Radial Flow)

  $H_V = ½\, H = 17.5'$

  $$T_V = t\frac{C_V}{H_V{}^2}$$

  $$T_V = (30)\frac{0.6}{17.5^2} = 0.06$$

  $U_V$ = 28% (Estimate from FHWA-RD-86-168; Vertical Flow)

  $U_C = 1.00[(1.00 – 0.64)(1.00 – 0.28)]$
  $U_C$ = 0.74 or 74%

  Settlement of layer 3 @ 74%
  $\Delta H = (0.74)(11.8'') = 8.7''$ @ 30 days

- Check for t = 68 days

  $T_R = t\, C_H/d_e^2 = (68)(0.6)/(9.5)^2 = 0.45.$

  $U_R$ = 90%
  $$T_V = t\frac{C_V}{H_V{}^2} = (68)\frac{0.6}{17.5^2}$$

  $U_V$ = 48%

  $U_C = 1.00[(1.00 – 0.90)(1.00 – 0.48)] = 0.95$

- Settlement of Layer 3 @ 95%
  $\Delta H = (0.95)(11.8'') = 11.2''$ @ 68 days

**Examine Option of Wick Drains – No Surcharge**

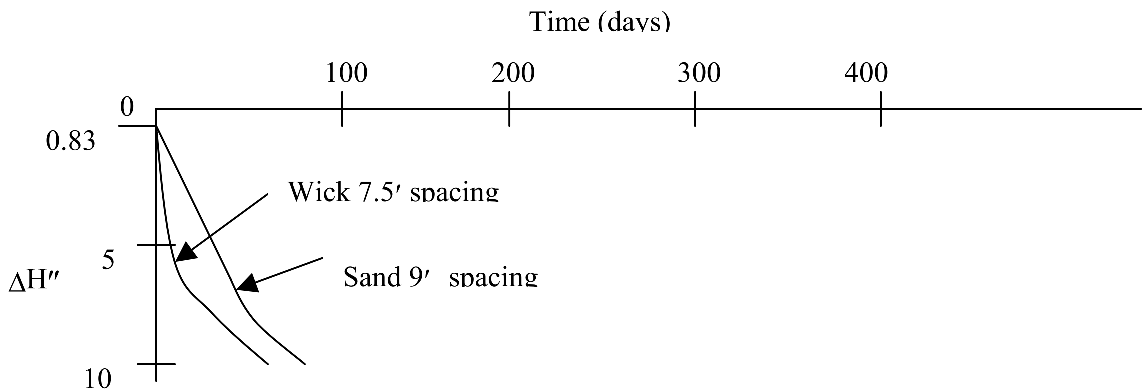
**Step 1: Assume equivalent sand drain diameter and perform analysis as for sand drains.**

Recent designs for wick drains have used equivalent diameters of 10 – 15 cm

$d_{equivalent}$ = 15 cm or 0.5′ = $d_w$
$C_v = C_{radial}$

Try 7.5′ center to center spacing triangular $d_e$ = 1.05(7.5′) = 7.9′

| $t_{days}$ | $U_V$% | $U_R$% | $U_C$% | **Layer 3 ΔH″** |
|---|---|---|---|---|
| 10 | 16 | 34 | 44 | 5.2 |
| 20 | 22 | 57 | 66 | 7.8 |
| 30 | 27 | 71 | 79 | 9.3 |
| 40 | 32 | 81 | 87 | 10.3 |
| 50 | 33 | 85 | 90 | 10.6 |
| 60 | 39 | 92 | 95 | 11.2 |




**Step 2: Prepare cost estimate for vertical drains**

Assume:

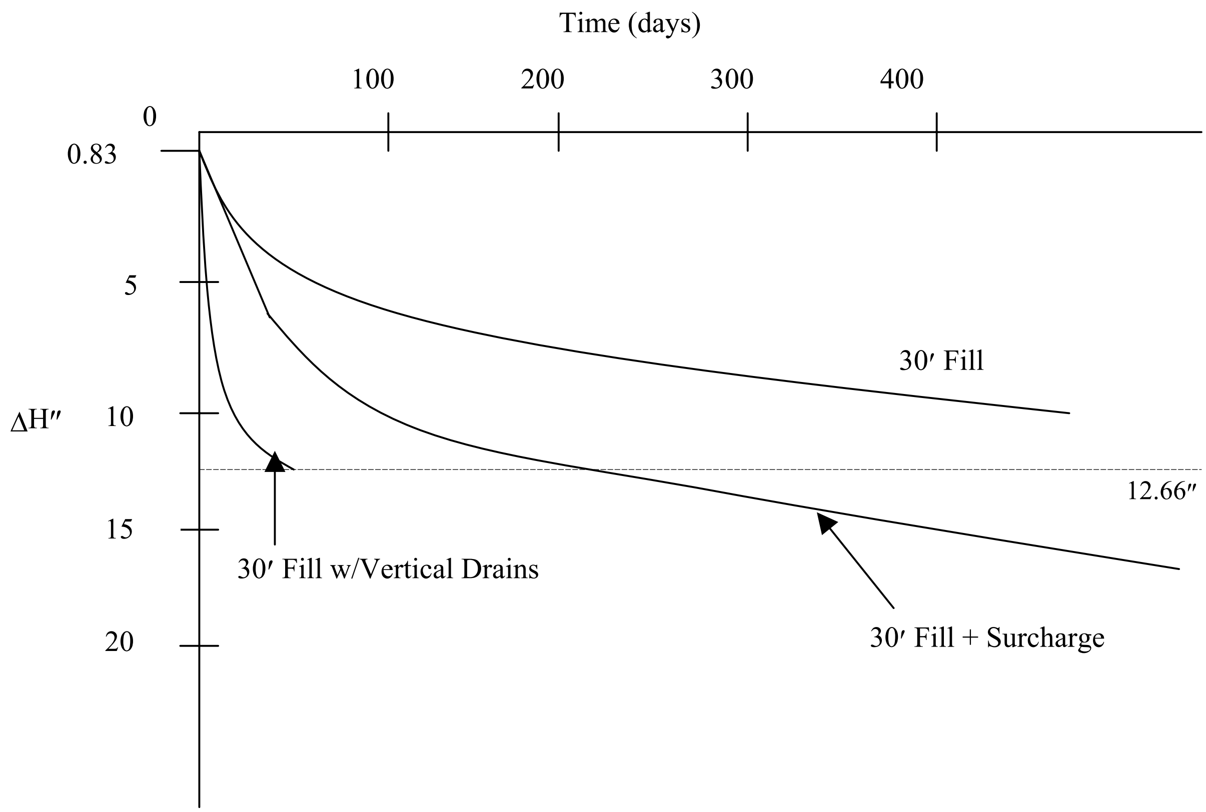
1. 500 L.F. of drains @ both approaches: Total 1000 L.F.
2. Width of drain treatment midslope to midslope: Total 160 L.F.
3. Length of drains: each 45 feet
4. Unit cost: per each – Sand \$3.50/ft
   Wick \$1.00/ft

- Sand Drains 9′ C – C

  Treated area/drain = 0.866 $S^2$ = 0.866$(9)^2$ = 70 S.F.
  No. of drains = (160)(1000)S.F./70 S.F. = 2286
  Linear feet of drain = (2286)45 = 102,870 L.F.
  Cost = (102,870)(\$3.50) + \$25,000(Mobilization) = \$385,000

  Estimated production at 1800 L.F. per day for 1 rig = 57 days construction time.

- Wick Drains 7.5′ C – C
  Treated area/drain = $(0.866)(7.5)^2$ = 49 S.F.
  No. of drains = 160,000/49 = 3265
  Linear footage = (3265)(45) = 146,925 L.F.
  Cost = (146,925)($1.00) + $25,000 (Mobilization) = $172,000




Time – settlement relationship for (a) 30′ fill, (b) 30′ fill with vertical drains and, (c) 30′ fill and surcharge

| **Treatment** | **t Months** | **Extra Cost** |
|---|---|---|
| Fill Only | 14 | - |
| Fill w/10′ Surcharge | 6 | $120,000 |
| Fill w/Wick Drains | 2 | $172,000 |
| Fill w/Sand Drains | 2 | $385,000 |

**Estimate amount of horizontal abutment movement due to lateral squeeze of clay.**

Rule of Thumb:

Horizontal Movement = 0.25 ΔH of embankment (in clay)
= $0.25 \times 11.8''$

Horizontal Movement = 3″

Recommend no spread footing construction or pile driving unit settlement nearly complete ($t_{90}$).

**Summary of the Embankment Settlement Phase for Apple Freeway Design Problem**

- Design Soil Profile

  Soil layer consolidation properties selected.

- Settlement

  32" of settlement predicted
  19.5" in organic
  0.8" in sand
  11.8" in clay.

- Time-Rate

  433 days for $T_{90.}$

- Surcharge

  10′ surcharge improves $t_{90}$ to 180 days
  cost $120,000.  F.S. w/ surcharge = 1.33 O.K.

- Vertical Drains

  60 days for $t_{90}$
  cost between $172,000 and $385,000

# CHAPTER 7.0
# SPREAD FOOTING DESIGN

The geotechnical design of a spread footing foundation is a two-part process. First the allowable soil bearing capacity must be established to insure stability of the footing and determine if the proposed structure loads can be supported on a reasonably sized footing. Second, the amount of settlement due to the actual structure loads must be predicted and time of occurrence estimated. Experience has shown that settlement is usually the controlling factor in the decision to use a spread footing foundation. This is not surprising as structural considerations usually limit tolerable settlements to values which can only be achieved on competent soils not prone to bearing capacity failure.

## 7.1 FOUNDATION DESIGN PROCEDURE

Foundation design is required for all structures to insure that the loads imposed on the underlying soil will not cause shear failures or damaging settlements. The duty of the foundation engineer is to establish the most economical design which safely conforms to prescribed structural criteria and properly accounts for the intended function of the structure. Essential to the foundation engineer's study is a rational method of design, whereby various foundation types are systematically considered and the optimum alternative selected. Indiscriminate selection of foundation type is verboten. Consideration of the following design approach will satisfactorily establish the proper type.

1. Determine the foundation loads to be supported and special constraints such as:

    a. Underclearance requirements which limit allowable total settlement.
    b. Structural design methodology which limits allowable differential settlement.
    c. Structural loads and tolerable deflections.
    d. Time constraints on construction.

    In general, a predesign discussion with the structural engineer will provide these answers and an indication of the degree of flexibility of the constraints.

2. Evaluate the subsurface data and laboratory testing with regard to reliability and completeness. The design method chosen should be commensurate with the quality and quantity of available geotechnical data, i.e., don't use state-of-the-art computerized analyses if you have not taken borings.

3. Consider alternate foundation types where applicable.

## 7.2 BEARING CAPACITY OF SPREAD FOOTINGS

Textbooks present varying theories and failure mechanisms for shallow footings. For the practicing engineer these theoretical discussions hold little interest. However, certain practical information can be drawn from the geometrics of the failure zone.

1. The bearing capacity of a footing is dependent on the strength of the soil within a depth below the footing of about 1 1/2 the footing width (unless much weaker soils exist just below this level).

Therefore, representative soil samples and frequent SPT values must be obtained in this zone. Continuous soil samples and SPT values should be routinely specified to a depth equal to twice the footing width. If the borings for a structure are done long before design, a good practice is to obtain continuous split spoon samples for the top 15 feet of each boring where footings may be placed on natural soil. The cost of this sampling is minimal but the knowledge gained is great including:

a. Thickness of existing topsoil.
b. Location of any thin zones of unsuitable material.
c. Accurate determination of depth of existing fill.
d. Improved ground water determination in the critical zone.
e. Representative samples in this critical zone to permit confident assessment of bearing capacity.

2. Often questions arise during excavation near existing footings as to the effect of soil removal on bearing capacity. In general, for weaker soils this zone extends outside the footing edge less than twice the footing width. Reductions in bearing capacity can be estimated by considering effects of removal within these zones. The lateral extent of this theoretical zone (Figure 7-1) is also useful in determining effects of ground irregularities on footing capacity or the effects of footing loads on adjacent facilities.

The general mechanism by which soils resist a footing load is similar to an embankment resisting shear failure. The load to cause failure must exceed the available soil strength on the failure plane and cause uplift of the weight of soil above the footing. When failure occurs the footing plunges into the ground and causes an uplift of soil adjacent to the sides of the footing.

The resistance to failure is based on the soil strength and amount of soil above the footing. The bearing capacity can be increased by:

1. Replacing or densifying the soil below the footing.

2. Increasing the embedment of the footing below ground.

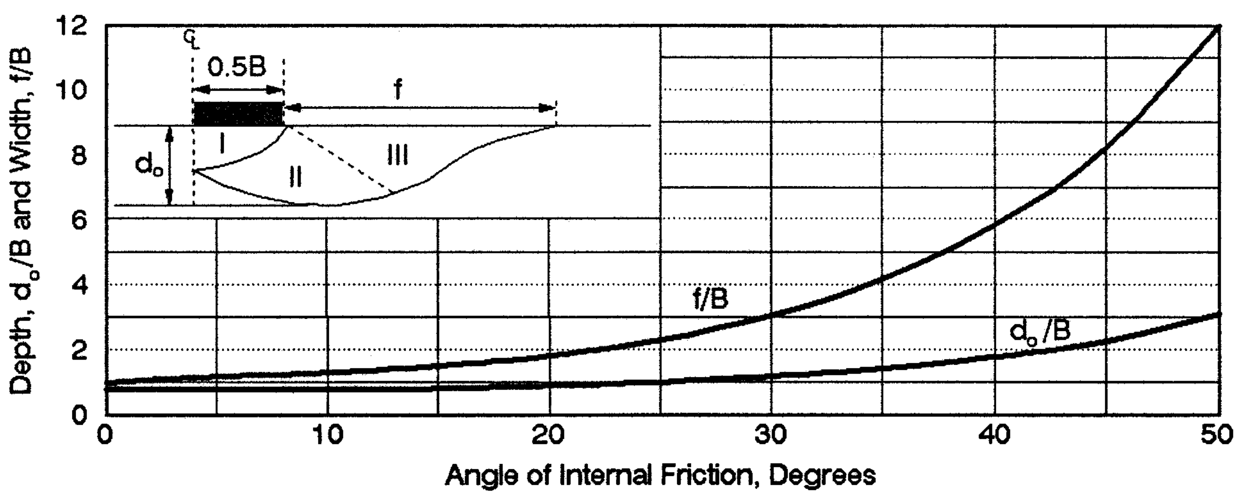



Figure 7-1: Variation of Depth ($d_0$) and Lateral Extent (f) of Influence of Footing with Angle of Friction

Common examples of improving bearing capacity are the support of temporary footings on pads of gravel or the embedment of mudsills a few feet below ground to support falsework. The design of these support systems is primarily done by bearing capacity analysis using the results of subsurface explorations and testing. Structural engineers who review falsework designs should carefully check the soil bearing capacity at foundation locations.

### 7.2.1 Bearing Capacity Computation

The procedure to be used to compute bearing capacity is as follows:

1. Review the structure plan to determine the proposed footing width. In the absence of data assume pier footing width equal to 1/3 the pier column height and abutment footing width equal to 1/2 the abutment height.

2. Review the soil profile to determine the position of the water table and the soil layer(s) which exist within the appropriate depth (1.5B) below the proposed footing level.

3. Review soil test data to determine the unit weight, friction angle and cohesion of the soils. In the absence of test data these values may be estimated for granular soils from standard penetration test data (Table 7-1) which has been corrected for overburden pressure. NOTE, the reliability of SPT values to determine shear strength of cohesive soils is poor. The SPT values in cohesive soils should not be used for determination of shear strengths for final design.

**TABLE 7-1**
**ESTIMATION OF SOIL PARAMETERS FROM STANDARD PENETRATION TESTS**

a. Granular Soil (Sand)

| **Description** | **Very Loose** | **Loose** | **Medium** | **Dense** | **Very Dense** |
|---|---|---|---|---|---|
| Standard penetration resistance corr'd, N'* | 0 | 4 | 10 | 30 | 50 |
| Approx. angle of internal friction, ($\phi$)degrees** | 25 – 30 | 27 – 32 | 30 – 35 | 35 – 40 | 38 – 43 |
| Approx. range of moist unit weight, ($\gamma$)pcf** | 70 – 100 | 90 – 115 | 110 – 130 | 120 – 140 | 130 – 150 |

* N' is SPT value corrected for overburden pressure.
** Use larger values for granular material with 5% or less fine sand and silt.

b. Cohesive soils (Clay) - (Rather unreliable, use only for preliminary estimate purposes).

| **Consistency** | **Very Soft** | **Soft** | **Medium** | **Stiff** | **Very Stiff** | **Hard** |
|---|---|---|---|---|---|---|
| $q_u$, ksf | 0 | 0.5 | 1.0 | 2.0 | 4.0 | 8.0 |
| Field standard penetration Resistance, N | 0 | 2 | 4 | 8 | 16 | 32 |
| $\gamma$(moist) pcf | 100 – 120 | 110 – 130 | | 120 – 140 | | |

4. Use the appropriate equation on Figures 7-2 through 7-5 to compute the ultimate bearing capacity. The continuous footing general case may be used when the footing length is 9 or more times the footing width. Also the bearing capacity factor $N_\gamma$ will usually be determined for a rough base condition since most footings are poured concrete. However the contact material smoothness must be considered for temporary footing such as wood grillages (rough), or steel supports (smooth) or plastic sheets (smooth). The safety factor for spread footing bearing capacity is selected both to limit the amount of soil strain and to account for variations in soil properties at footing locations.

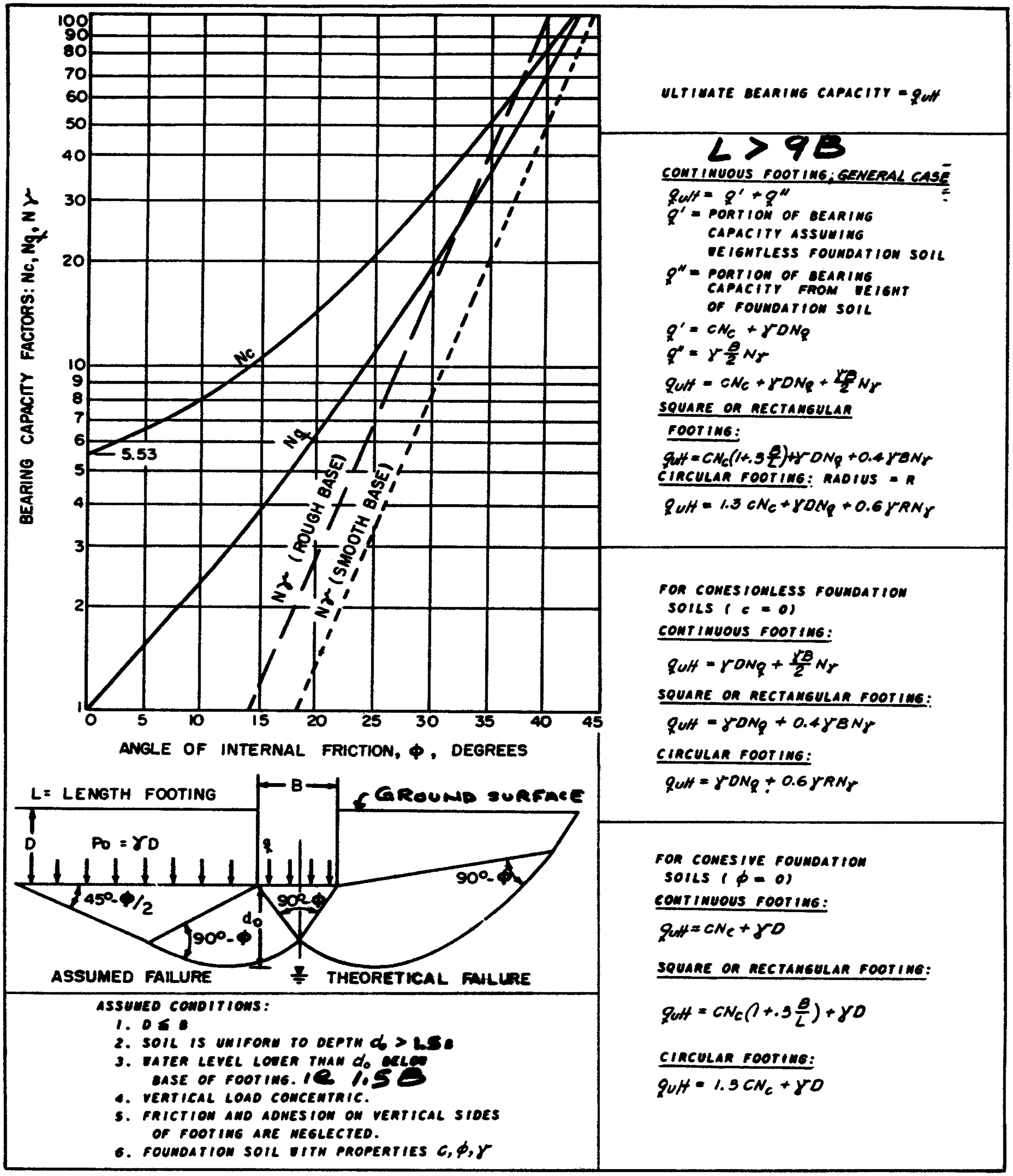
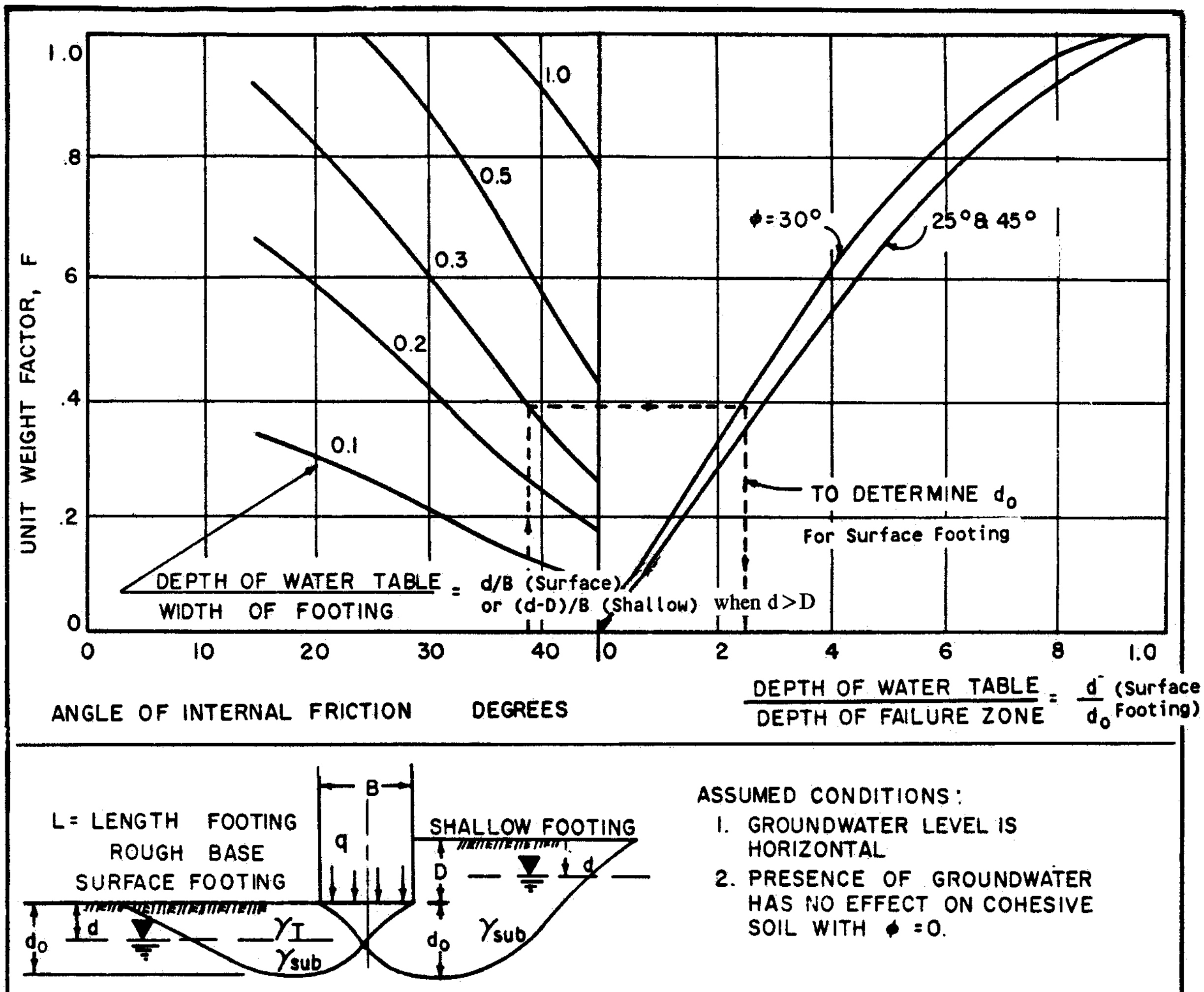



Figure 7-2: Ultimate Bearing Capacity of Shallow Footings with Concentric Loads




**CONTINUOUS FOOTING:**

**SURFACE FOOTING: D = 0**

$$q_{ult} = cN_c + \left[\gamma_{sub} + F(\gamma_T - \gamma_{sub})\right] \frac{B}{2} N_\gamma$$

**SHALLOW FOOTING: $D \leq B$**

IF $d \leq D$

$$q_{ult} = cN_c + \left[\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d\right] N_q + 0.5 \gamma_{sub} B N_\gamma$$

IF $D < d \leq (D + d_0)$

$$q_{ult} = cN_c + \gamma_T D N_q + \left[\gamma_{sub} + F(\gamma_T - \gamma_{sub})\right] \frac{B}{2} N_\gamma$$

VALUES OF BEARING CAPACITY FACTORS $N_c$ $N_q$ AND $N_\gamma$ ARE SHOWN IN FIGURE 14

**RECTANGULAR FOOTING:**

**SURFACE FOOTING: D = 0**

$$q_{ult} = cN_c\left(1 + .3\frac{B}{L}\right) + \left[\gamma_{sub} + F(\gamma - \gamma_{sub})\right] 0.4 B N_\gamma$$

**SHALLOW FOOTING: $D \leq B$, IF $d \leq D$**

$$q_{ult} = cN_c\left(1 + .3\frac{B}{L}\right) + \left[\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d\right] N_q + 0.4 \gamma_{sub} B N_\gamma$$

IF $D < d \leq (D + d_0)$

$$q_{ult} = cN_c\left(1 + .3\frac{B}{L}\right) + \gamma_T D N_q + \left[\gamma_{sub} + F(\gamma_T - \gamma_{sub})\right] 0.4 B N_\gamma$$

**CIRCULAR FOOTING: RADIUS = R = B/2**

**SURFACE FOOTING: D = 0**

$$q_{ult} = 1.3 cN_c + \left[\gamma_{sub} + F(\gamma_T - \gamma_{sub})\right] 0.6 R N_\gamma$$

**SHALLOW FOOTING: $D \leq 2R$, IF $d \leq D$**

$$q_{ult} = 1.3 cN_c + \left[\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d\right] N_q + \left[0.6 \gamma_{sub} \cdot R N_\gamma\right]$$

IF $D < d \leq (D + d_0)$

$$q_{ult} = 1.3 cN_c + \gamma_T D N_q + \left[\gamma_{sub} + F(\gamma_T - \gamma_{sub})\right] 0.6 R N_\gamma$$

Figure 7-3: Ultimate Bearing Capacity with Ground Water Effect

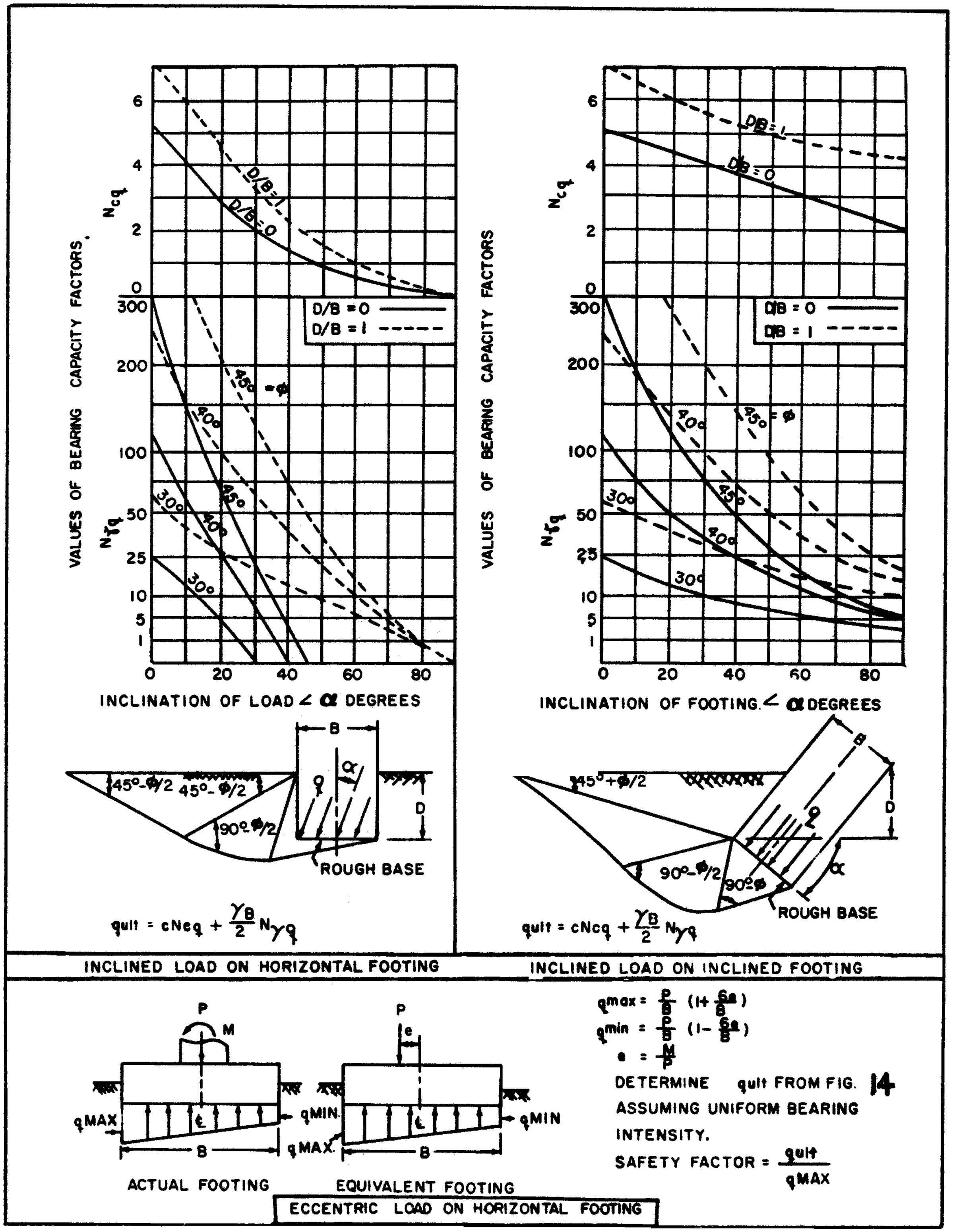
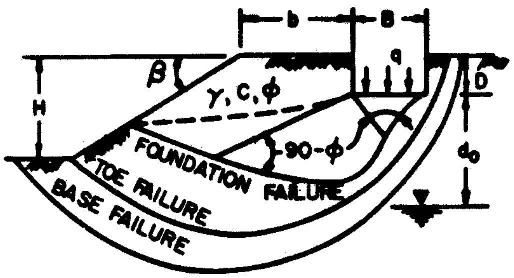



Figure 7-4: Ultimate Bearing Capacity Continuous Footing with Eccentric or Inclined Loads

## CASE I: CONTINUOUS FOOTING AT TOP OF SLOPE




Water at $d_o \geq B$

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \qquad ①$$

Water at Ground Surface

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \qquad ②$$

If $B \leq H$:

Obtain $N_{cq}$ from Figure 17B for Case I with $N_o = 0$.
Interpolate for values of $0 < D/B < 1$

Interpolate $q_{ult}$ between EQ ① and ② for water at intermediate level between ground surface and $d_o = B$.

If $B > H$:

Obtain $N_{cq}$ from Figure 17B for Case I with stability number

$$N_o = \frac{\gamma H}{C}.$$

Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.

Interpolate $q_{ult}$ between EQ ① and ② for water at intermediate level between ground surface and $d_o = B$. For water at ground surface and sudden drawdown: substitute $\phi'$ for $\phi$ in EQ ②

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T} \tan \phi\right)$$

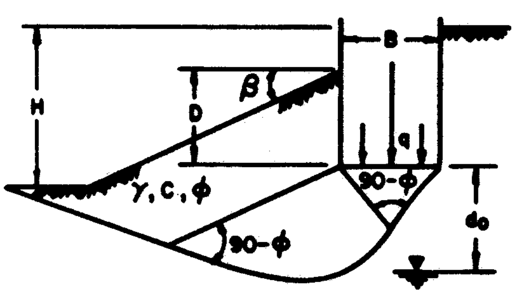
Cohesive soil ($\phi = 0$)

Substitute in EQ ① and ② D for $B/2$ and $N_{\gamma q} = 1$.

Rectangular, square or circular footing:

$$q_{ult} = \left[q_{ult} \text{ for continuous footing as given above}\right] \times \left[\frac{q_{ult} \text{ for finite footing}}{q_{ult} \text{ for continuous footing}}\right] \text{ from Fig. 14}$$

## CASE II: CONTINUOUS FOOTINGS ON SLOPE




Same criteria as for Case I except that $N_{cq}$ and $N_{\gamma q}$ are obtained from diagrams for Case II

Figure 7-5A: Ultimate Bearing Capacity for Shallow Footing Placed on or Near a Slope

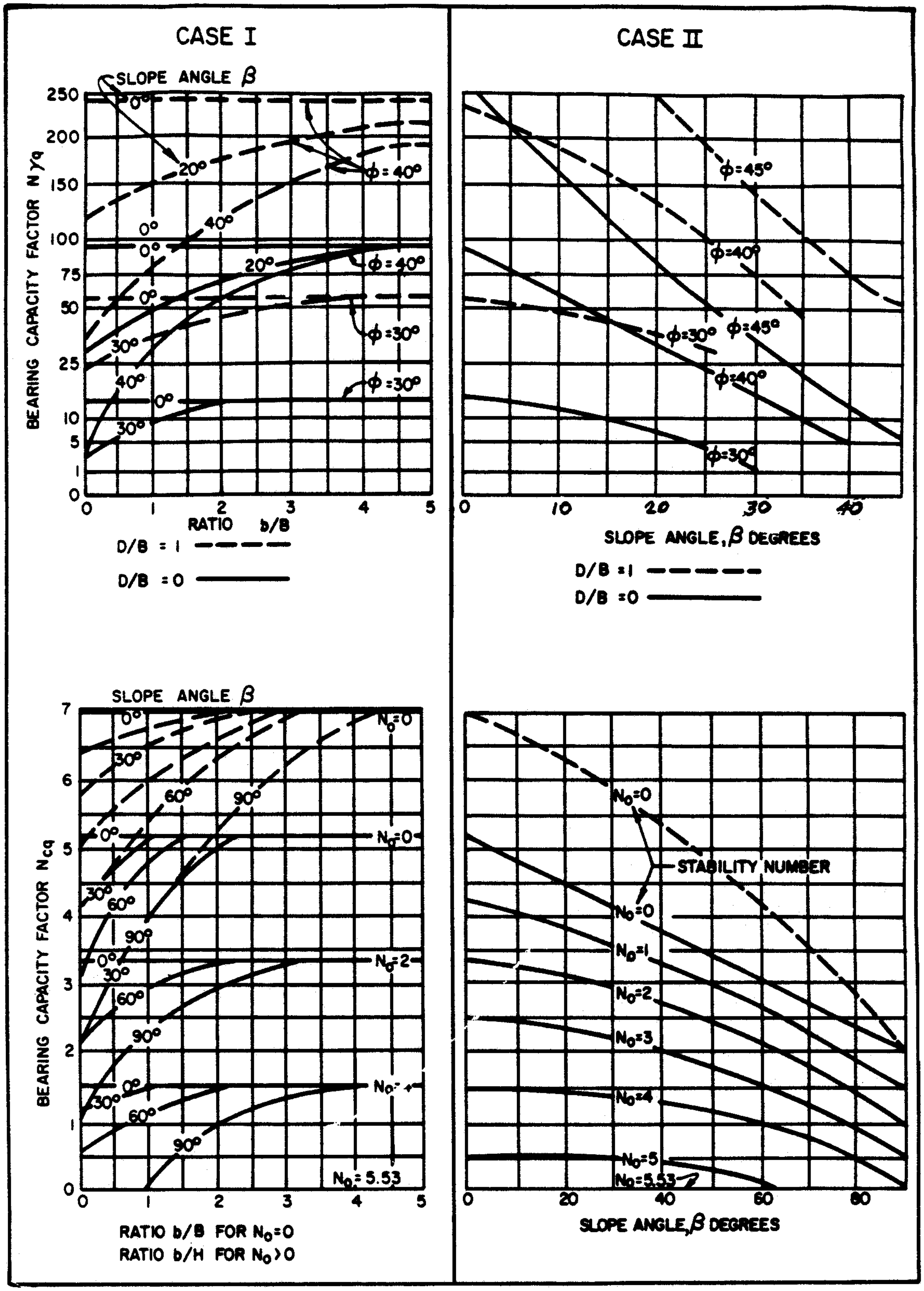



Figure 7-5B: Bearing Capacity Factors for Shallow Footing Placed on or Near A Slope

### 7.2.2 Practical Aspects of Bearing Capacity Computations

Many footings are designed structurally to resist a combination of vertical and horizontal loads which produce a trapezoidal load distribution under the footing. In sizing a footing to accommodate a recommended maximum allowable bearing capacity, the question arises on whether the recommended value refers to the average value across the footing or the maximum value under the footing edge. Satisfactory results may be obtained by sizing the footing so that the average pressure does not exceed the recommended value and the maximum edge pressure does not exceed 1.3 times the average value.

The effect of a high ground water table on the bearing capacity of a footing is frequently over-estimated. Some textbooks and public codes mandate reduction of the allowable bearing capacity by one half if the ground water is within a depth of one footing width below the footing. Such a large reduction only applies if the design ground water level is at or above ground surface in granular soils. The only soil element affected by ground water is the unit weight ($\gamma$). An examination of the general bearing capacity equation indicates two of the three terms include soil unit weight. One term refers to the amount of soil above the footing; the other to soil below the footing. As water rises up toward the footing level, only one factor is reduced. For complete reduction by half, the water must rise above the ground. The effect of high ground water on bearing capacity is accurately taken into account in Figure 7-3.

The general effects of changes in either soil properties or footing dimensions on bearing capacity need to be understood. The general equation for bearing capacity is:

$$q_{ult} = cN_c + \gamma DN_q + 1/2\gamma BN_\gamma \qquad (7\text{-}1)$$

Note first that bearing capacity is composed of separate contributions from the soil's cohesive strength, the embedment depth of the footing, and the soil's frictional strength. Table 7-2 shows how bearing capacity can vary with changes in physical properties or dimensions.

**TABLE 7-2**
**VARIATION IN BEARING CAPACITY WITH CHANGES IN PHYSICAL PROPERTIES OR DIMENSIONS**

| Properties and Dimensions | | Cohesive Soil | Cohesionless Soil |
|---|---|---|---|
| $\gamma$ = Unit Weight<br>D = Footing Embedment<br>B = Footing Width | | $\phi = 0$<br>c = 1000 psf<br>$q_{ult}$ (psf) | $\phi = 30^o$<br>c = 0<br>$q_{ult}$ (psf) |
| A. | Initial situation $\gamma$ = 120 pcf, D = 0', B = 5' Deep water table | 5530 | 5400 |
| B. | Effect of embedment D = 5', $\gamma$ = 120 pcf, B = 5', deep water table | 6130 | 17400 |
| C. | Effect of width, B = 10' $\gamma$ = 120 pcf, D = 0', deep water table | 5530 | 10800 |
| D. | Effect of water table at surface $\gamma$ = 57.6 pcf, D = 0', B = 5' | 5530 | 2592 |

Notice that the effect of the variables on the bearing capacity in cohesive soils is minimal. Only the embedment has an effect on bearing capacity. Also note that the water table rise does not influence cohesion. Interparticle bonding will remain unchanged unless the clay contains minerals which react to water immersion, i.e., expansive minerals, or the clay is reworked.

Notice the effect on cohesionless soils is great when properties and dimensions are changed. The embedment effect is particularly important. Removal of soil from over an embedded footing, either by excavation or scour, can substantially reduce bearing capacity and cause footing subsidence. Rehabilitation or repair of existing spread footing often requires excavation of the soil above the footing. If the effect of this removal on bearing capacity is not considered, the footing may move downward; resulting in structural distress.

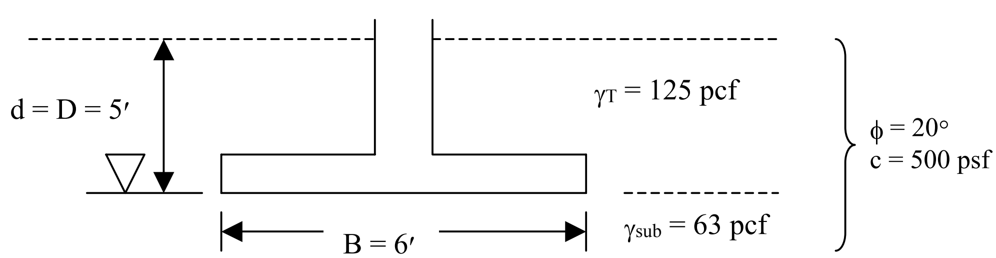
Two modes of bearing capacity failure exist: general shear failure and local shear failure. Local shear failure is characterized by a "punching" of the footing into the ground when weak soils exist below footing level or when very narrow footings are used. This local condition seldom applies to bridge structures because spread footing are not used on obviously weak soils and relatively large footing sizes are needed for structural stability.

The mechanism of general bearing capacity failure is similar to the embankment failure mechanism. However, the footing analysis is a 3-dimensional analysis as opposed to the 2-dimensional slope stability analysis. The bearing capacity factors $N_c$, $N_q$ and $N_\gamma$ relate to the actual volume of soil involved in the failure. A cursory study of the footing failure cross section in Figure 7-2, discloses that the depth and lateral extent of the failure (and therefore the value of $N_c$, $N_q$ and $N_\gamma$) is determined by the dimensions of the wedge-shaped zone directly below the footing. As the friction angle increases, the depth and width of the failure zone increase; thus mobilizing more soil shear strength and increasing the bearing capacity.

Substantial downward movement of the footing is required to completely mobilize the shearing resistance along the entire failure surface. For this reason the safety factor which is used to find allowable bearing capacity is composed of two partial safety factors; a factor of 2.0 to limit strain and a factor of 1.25 - 1.50 for uncertainties in soil information. The total safety factor is usually 3 if standard penetration values were used to determine strength properties of the soil. This large safety factor insures that only minimal movement (strain) is necessary to fully mobilize the allowable bearing capacity.

Lastly, in reporting the results of bearing capacity analyses, always include the footing width that was used to compute the bearing capacity. Most often the geotechnical engineer must assume a footing width as bearing capacity analyses are completed before structural design begins. It is recommended that bearing capacity be computed for a range of possible footing widths and those values be included in the foundation report with a note stating that if other footing widths are used, the geotechnical engineer should be contacted. Remember that changes in footing width cause large changes in unit bearing capacity in granular soils.

**Example 7-1**: Determine the Allowable Bearing Capacity for a Rough Base Square Footing Using a Safety Factor of 3.



**Solution:**

Assuming a general shear condition, enter the bearing capacity chart for $\phi = 20°$ and read $N_c = 14$, $N_q = 6$, $N\gamma = 3$. Also note that formula for bearing capacity must account for the square footing and the water table within the failure zone.

$$q_{ult} = (1 + 0.3\frac{B}{L})cN_c + [\gamma_{sub} D + (\gamma_T - \gamma_{sub})d] N_q + 0.4\gamma_{sub} BN_\gamma \tag{7-2}$$

$$= (1.3)(500)14 + [63(5) + (125 - 63)5]6 + 0.4(63)(6)(3)$$

$$= 9100 + 3750 + 450$$

$$q_{ult} = 13,300 \text{ psf}$$

$$q_{all} = \frac{q_{ult}}{3} = \frac{13,300}{3} \cong 4,430 \text{ psf}$$

### 7.2.3 Spread Footing Load Tests

Spread footing load tests can be used to verify both bearing capacity and settlement predictions. Full scale tests have been done on predominantly granular soils. An example is the I-359 project in Tuscaloosa, Alabama where dead load was placed on 12' x 12' footings to create a foundation contact pressure of over 4 tsf. The greatest settlement recorded was about 0.1 inches. (An additional 0.1 inch was recorded when the footing concrete was placed).

A new dynamic procedure called the WAK test, is also available to assess the stiffness of soils below footings (ASCE Journal of Geotechnical Engineering, Vol. 116, No. 3, March 1990).

### 7.2.4 Computer Program

FHWA has funded the development of a user friendly computer program, CBEAR. The users manual is FHWA-TA-91-047, "CBEAR - Bearing Capacity Analysis of Shallow Foundations." The major use of this program is to compute bearing capacity of footings with complex loading conditions.

## 7.3 SETTLEMENT OF SPREAD FOOTINGS

The controlling factor in the design of a spread footing foundation is usually tolerable settlement. Prediction of settlement may be routinely accomplished with adequate geotechnical data and a knowledge of the proposed structure loads. The accuracy of the prediction is only as good as the quality of the geotechnical data and the estimation of the actual loads placed on the footing. Settlements of spread footings are frequently overestimated by engineers for the following reasons:

1. The structural load (P) causing the settlement is overestimated. In the absence of actual structural loads, geotechnical engineers conservatively assume that the footing pressure equals the maximum allowable soil bearing pressure.

2. Settlement occurring during construction is not subtracted from total predicted amounts.

3. Preconsolidation of the subsoil is not accounted for in the analysis. This preconsolidation may be due to a geologic load applied in past time or to removal of significant amounts of soil in construction previous to placing the foundation. This error can cause a grossly overestimated settlement.

To rationally predict settlement of spread footings, the following procedure should be used:

### 7.3.1 General Procedures for Both Cohesionless and Cohesive Soils

1. Plot soil profile including soil unit weights, consolidation test values for design and SPT results (N).

2. Draw existing effective overburden pressure diagram ($P_o$) with depth.

3. Plot design bearing pressure on $P_o$ diagram at proper footing level.

4. Distribute design bearing pressure with depth by 2 on 1 method or other appropriate distribution method.

   2:1 Pressure Distribution Method:

   If footing is continuous, $L \geq 9W$; $\Delta P = \frac{W}{W + X}(P)$ (7-3a)

   If footing is rectangular, $L < 9W$; $\Delta P = \frac{WL}{(W + X)(L + X)}(P)$ (7-3b)

   Where:
   X = depth below footing
   W = footing width
   L = footing length
   P = applied footing pressure

   a. Project 2 vertical on 1 horizontal lines down from footing corners. Compare original footing area to area generated on a plane at various depths below the footing, i.e., if a 10′ by 40′ footing is loaded to 2000 psf, the pressure in the ground at 20′ below footing level is:

   $$\Delta P = \frac{10 \times 40\ (2000)}{(10 + 20)\ (40 + 20)} = 444 \text{ psf}$$

5. Extend footing pressure distribution at least to a level where the distributed footing pressure ($\Delta P$) is 1/10 of the overburden pressure at that depth. This depth is commonly referred to as the critical depth.

### 7.3.2 Settlement Computation for Cohesionless Soils

Settlement of granular soils is usually elastic and consolidation occurs immediately on application of load.

1. Determine corrected SPT value (N′) from Figure 6-5.

2. Determine bearing capacity index (C′) by entering Figure 6-6 with N′ value from (1).

3. Compute settlement in 10′ ± increments of depth from

$$\Delta H = H \frac{(1)}{C}, Log \frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}1)$$

Where:
$\Delta H$ = Settlement
$H$ = Thickness of soil layer considered
$C'$ = Bearing capacity index
$P_o$ = Existing effective overburden pressure at center of considered layer (psf)
$\Delta P$ = Distributed footing pressure at center of considered layer (psf)
$P_F$ = $P_o + \Delta P$

4. Studies conducted by FHWA indicated that this procedure is conservative and will over-predict the settlement by a factor of about 2.

### 7.3.3 Engineering Practice - Settlement and Differential Settlement (see publication FHWA-RD-86-185 for details)

A common practice for predicting settlement of footings on sand is to use one or more of the available calculation methods i.e., Hough, Peck-Bazaraa, D'Appolonia, Schmertmann. Engineering judgment is then used to select one of the results, or average the results, based on the appropriate approach. Experience has shown that structure foundations consisting of footings designed in this manner have a very high probability of acceptable performance.

A practical method for calculating differential settlement between adjacent footings on sand involves one or more of the following concepts:

1. If borings are performed at each footing location, calculate the differential settlement as the difference in the estimated total settlement of each footing, calculated based on the individual borings.

2. Lesser amounts of boring data only permit empirical estimates such as suggested by Terzaghi and Peck (1967), if footings are about the same length and width, calculate maximum differential settlement as 50 percent of the maximum total settlement. If footings are of different sizes, calculate differential as 75 percent of the maximum total value.

3. If the penetration resistance of the soil is highly variable from boring to boring, calculate maximum differential settlement as 100 percent of the maximum total settlement.

### 7.3.4 Settlement Computation for Cohesive Soils

Settlement of spread footings on cohesive soils is usually due to primary compression as spread footings are usually not placed on soils with significant secondary compression characteristics.

1. Analyze consolidation test data to determine:

   a. Preconsolidation pressure ($P_c$)

   b. Initial void ratio ($e_o$) at $P_o$

   c. Compression and recompression indices ($C_c$ & $C_r$)

1. (ALT) In the absence of consolidation test data, settlement may be approximated from Atterberg limit and moisture content data as follows in (a) through (c). This method is only recommended for use in design for soils not conducive to consolidation testing.

   a. Soil may be assumed to be preconsolidated to pressures above typical loads if the liquidity index ([moisture content minus plastic limit] divided by plastic index) is less than 0.7.

   b. Initial void ratio is determined for saturated soils by multiplying the specific gravity times the moisture content divided by 100.

   c. $C_c$ and $C_r$ are determined by dividing the moisture content by 100 and 1000 respectively.

2. Compute settlement in 10′ ± increments of depth or at soil layer boundaries from

$$\Delta H = H \frac{C_c}{1+e_0} \text{Log} \frac{P_F}{P_0} \quad \text{(For normally consolidated soils only)} \qquad (6\text{-}2a)$$

3. Compute time for settlement from $t = (TH_v^2) / C_v$, following the procedure shown for embankment settlement in the previous chapter.

These settlement analyses may be varied by the foundation engineer to determine:

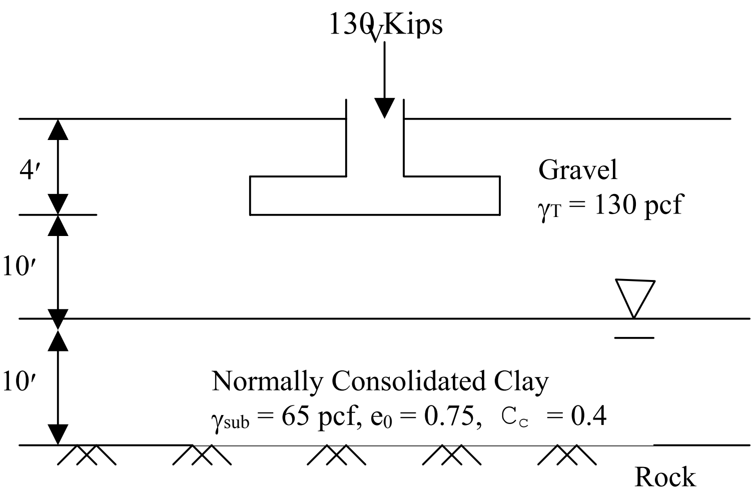
1. Percentage of settlement due to dead and live loads.

2. Effects of adjacent fill placement.

3. Footing width required to limit settlement to a tolerable value.

4. Amount of preload needed to reduce subsequent structure settlement.

**Example 7-2:** Determine the Settlement Of the 10′ × 10′ Square Footing Due To A 130 Kip Axial Load. Assume The Gravel Layer Is Incompressible.




**Solution:**

Find Overburden Pressure, $P_0$, at center of Clay Layer
$P_0 = (14' \times 130 \text{ pcf}) + (5' \times 65 \text{ pcf}) = 2{,}145 \text{ psf}$

Find Change in Pressure ()P) at Center of Clay Layer Due to Applied Load.

$$\Delta P = \frac{130 \text{ Kips}}{(10+15)^2} = \frac{130 \text{Kips}}{625} = 208 \text{ psf}$$

Find Settlement

$$\Delta H = H \frac{C_c}{1+e_0} \text{Log} \frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}2)$$

$$= 10\left(\frac{0.4}{1+0.75}\right) \text{Log} \frac{2145 \text{ psf} + 208 \text{ psf}}{2145 \text{ psf}}$$

$$\Delta H = 0.09' = 1.1''$$

## 7.4 SPREAD FOOTINGS ON EMBANKMENTS

One of the most basic conclusions established by foundation engineers was the desirability of placing footings on controlled fills. In general, the fill weight is many times the imposed footing load. If adequate time is allowed for the foundation soil to consolidate under the fill load, subsequent application of the smaller structure load will result in negligible structure settlement. In bridge construction, common practice is to build the approach embankment excluding the area to be occupied by the abutment and allow settlement to occur prior to abutment construction.

Field evaluation of spread footings placed in compacted embankments, constructed of select granular material, have shown that spread footings will provide satisfactory performance. A 1978 performance evaluation (FHWA RD-81/184) was conducted through a joint study between FHWA and the Washington State Highway Department. A visual inspection was made of the structural condition of 148 highway bridges supported by spread footings on compacted fill throughout the State of Washington. The approach pavements and other bridge appurtenances were also inspected for damage or distress that could be attributed to the use of spread footings on compacted fill. This review, in conjunction with detailed survey investigations of the foundation movement of 28 selected bridges, was used to evaluate the performance of spread footings on compacted fills. The study concluded that spread footings can provide a satisfactory alternative to piles especially when high embankments of good quality borrow materials are constructed over satisfactory foundation soils. None of the bridges investigated displayed any safety problems or serious functional distress. All bridges were in good condition. In addition to the performance evaluation, cost analyses and tolerable movement correlation studies were made to further substantiate the feasibility of using spread footings in lieu of expensive deep foundation systems. Cost analyses showed spread footings were 50 to 65 percent cheaper than the alternate choice of pile foundations. Foundation movement studies showed that these bridges have easily tolerated differential settlements of 1 to 3 inches without serious distress. A second nationwide study of 314 bridges (FHWA RD-85/107) arrived at similar conclusions. Unfortunately many agencies continue to disregard spread footing foundation alternates for highway structures. In NCHRP Synthesis 107 "Shallow Foundations For Highway Structures" the author concisely summarizes the chapter on performance criteria as follows:

"It is very clear that the tolerable settlement criteria currently used by most transportation agencies are extremely conservative and are needlessly restricting the use of spread footings for bridge foundations on many soils. Angular distortions of 1/250 of the span length and differential vertical movements of 2 to 4 inches, depending on span length, appear to be acceptable, assuming that approach slabs or other provisions are made to minimize the effects of any differential movements between abutments and approach embankments. Finally, horizontal movements in excess of 2 inches appear likely to cause structural distress. The potential for horizontal movements of abutments and piers should be considered more carefully than is done in current practice."

## 7.5 APPLE FREEWAY DESIGN EXAMPLE – SPREAD FOOTING DESIGN

In this chapter the Apple Freeway is used to illustrate the design process for spread footings for the pier and abutment. The computation process for evaluation of bearing capacity and settlement analysis are presented.

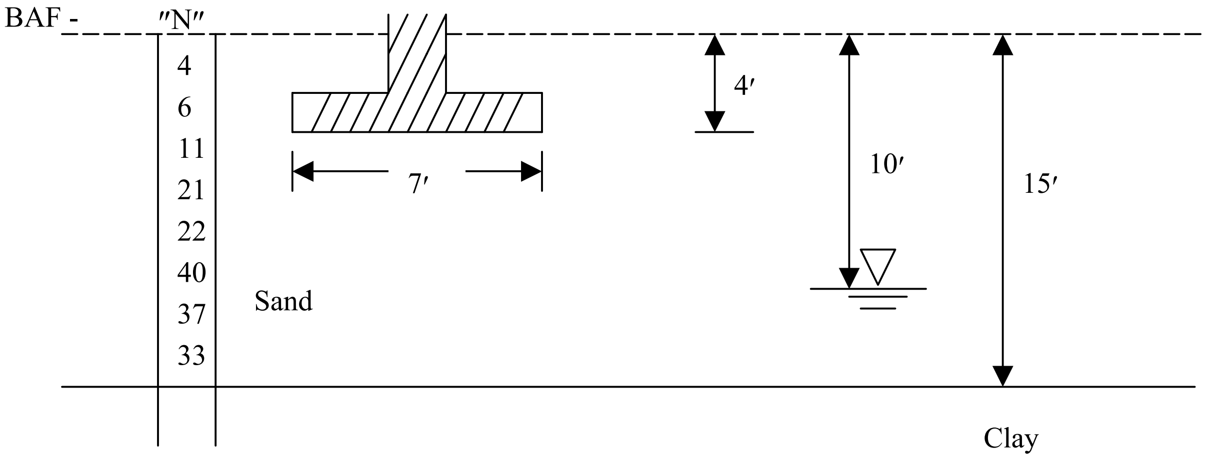
| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | Po Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| **Spread Footing Design** | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>    Pipe Pile<br>    H – Pile<br>Static Analysis – abutment<br>    Pipe Pile<br>    H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

Apple Freeway Design Example – Spread Footing Design
Exhibit A

**Pier Footing**

**Given:** The footing geometry and subsurface condition shown below.

**Required:** Compute the allowable bearing capacity, anticipated settlement and settlement rates for the pier footing.




Assumptions:

- Footing embeded 4′ below ground
- Footing width = 1/3 pier height = 7′
- Footing length = 100′
  L/W = 100/7 > 9 ∴ Continuous
- Water level 6′ below Footing (< 1.5B)
- Use SPT values to find $\phi$

## Compute Allowable Bearing Capacity

**Step 1: Find N′ below footing (use Figure 6-5 to obtain N′/N).**

| Depth | $P_0$ (psf) | N (bpf) | N′/N | N′ |
|---|---|---|---|---|
| 5 | 550 | 11 | 1.9 | 21 |
| 7 | 770 | 21 | 1.55 | 33 |
| 8 | 880 | 22 | 1.45 | 32 |
| 10 | 1100 | 40 | 1.27 | 51 |
| 12 | 1195 | 37 | 1.19 | 44 |
| 14 | 1290 | 33 | 1.12 | 37 |
| | | | Avg. N′ = 36 ∴ $\phi \approx 36°$ | |

**Step 2: Determine ultimate capacity ($Q_{ult}$).**

$$Q_{ult} = \cancelto{0}{cN_c} + \gamma_T DN_q + [\gamma_{sub} + F(\gamma_T - \gamma_{Sub})]\ \frac{B}{2} N\gamma$$

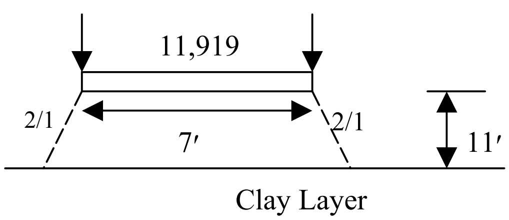
$$= (110)(4)(40) + [47.6 + \underbrace{(0.9)}_{F @ \frac{d-D}{B} \text{ of } 0.86 = 0.9 \text{ (Figure 7-3)}}(62.4)]\ (\frac{B}{2})\ 50$$ ($N_q$ & $N_\gamma$ from Figure 7-2)

$Q_{ult}$ = 17,600 + 18,158 = 35,758 psf

**Step 3: Determine allowable bearing capacity (use F.S. = 3)**

$$Q_{all} = \frac{35{,}758}{3} = 11{,}919 \text{ psf or} \sim 6 \text{ tsf}$$

**CHECK PRESSURE TRANSMITTED TO CLAY LAYER VERSUS ALLOWABLE CLAY BEARING CAPACITY**




**Step 1: Determine pressure on clay layer**

$$\text{Pressure @ clay surface} = \left(\frac{7}{7+11}\right)(11{,}919\ psf)$$

$P_{clay}$ = 4,635 psf

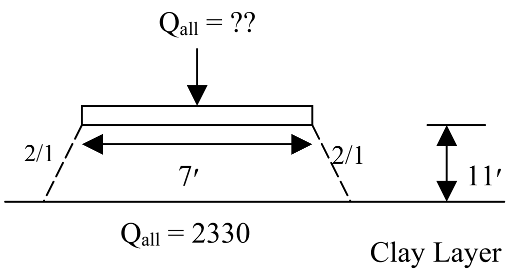
**Step 2: Check pier bearing capacity**

Pier Bearing Capacity at Top of Clay Layer

$$Q_{ult.clay} = cN_c + P_0 N_q$$

$$= (1100)(5.14) + (1338)(1) = 6992 \text{ psf}$$

$$Q_{all\ clay} = \frac{6992}{3} = 2330 \text{ psf}$$




Need to reduce footing pressure as $Q_{all\ clay} < P_{clay}$

$$Q_{all\ max} = Q_{all\ clay}\ \left(\frac{7+11}{7}\right)$$

$$Q_{all\ max} = 2330\ \left(\frac{18}{7}\right) = 5990\ \text{psf}$$

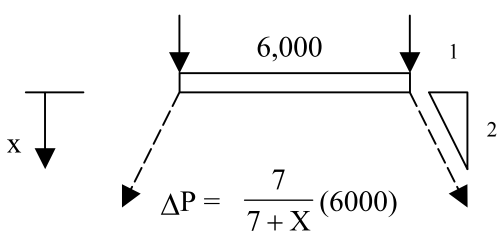
$Q_{all\ max}$ = 5,990 psf $\cong$ 3 tsf

Check w/ bridge designer to see if 3 tsf is a realistic pressure. Designer estimates a max. structure load of 2200 tons, and a minimum footing width of 7′ $\therefore$ Est. footing pressure (max) = $\dfrac{2200}{7\times 100}$

$Q_{FTS} \cong 3.1$ tsf
Use Q = 3 tsf for Settlement Analysis

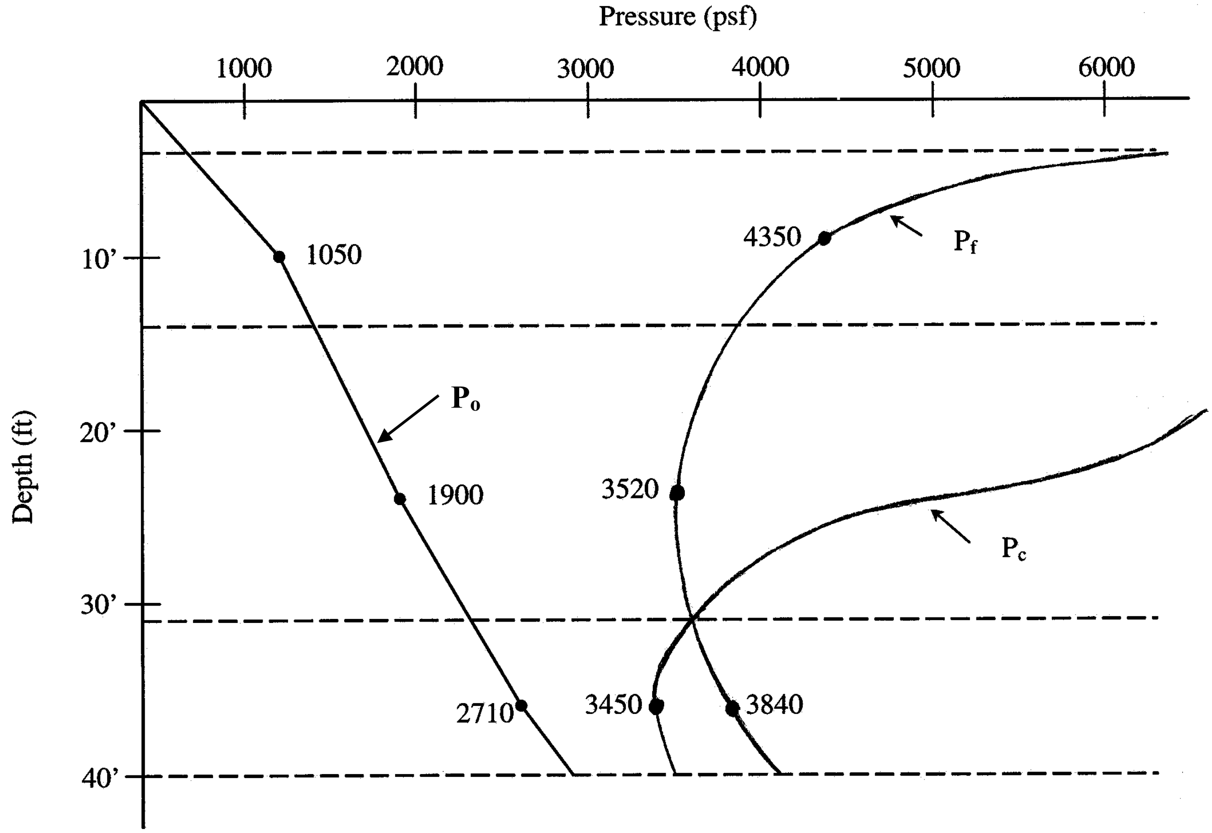
**Check Settlement**

**Step 1: Find pressure distribution by 2 on 1.**




| **Depth X** | $\mathbf{k = \frac{7}{7+x}}$ | **ΔP (psf)** |
|---|---|---|
| 3.5 | 0.67 | 4000 |
| 7 | 0.50 | 3000 |
| 10.5 | 0.40 | 2400 |
| 14 | 0.33 | 2000 |
| 21 | 0.25 | 1500 |
| 28 | 0.20 | 1200 |
| 35 | 0.17 | 1000 |

**Step 2:** **Plot curve of $P_o$, $P_c$ and $P_F$ ($P_o + \Delta P$) vs. depth (use to obtain pressure of layer center).**




**Step 3:** **Compute settlement in each layer.**

Pier Settlement

- Sand Layer 4′ - 15′

$$\Delta H = H\frac{1}{C'}\text{Log}\frac{P_F}{P_0}$$

N′avg = 36

C′ = 90

$$\Delta H = 11\frac{1}{90}\text{Log}\frac{4350}{1050}(12)$$

$\Delta H = 0.90''$

- Clay Layer 15′ - 32′

$P_0 \rightarrow P_C$ (Preconsolidated)

$$\Delta H = H\frac{C_R}{1+e_0}\text{Log}\frac{P_F}{P_0}$$

$$\Delta H = 17\frac{0.035}{1+0.97}\text{Log}\frac{3520}{1900}(12)$$

$\Delta H = 0.97''$

- Clay Layer 32′ - 40′

$P_0 \rightarrow P_C$ (Preconsolidated)

$$\Delta H = H\frac{C_R}{1+e_0}\text{Log}\frac{P_c}{P_0}$$

$$\Delta H = 8\frac{0.035}{1+0.97}\text{Log}\frac{3450}{2710}(12)$$

$\Delta H = 0.18''$

$P_c \rightarrow P_F$ (Not Preconsolidated)

$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_c}$$

$$\Delta H = 8\frac{0.35}{1+0.97}\text{Log}\frac{3840}{3450}(12)$$

$\Delta H = 0.80''$

| **Total Settlement** | |
|---|---|
| Sand | 0.90 |
| Clay | 0.97 |
| | 0.18 |
| | 0.80 |
| $\Delta H =$ | 2.85″ |

**Obtain Time-Settlement Relationship**

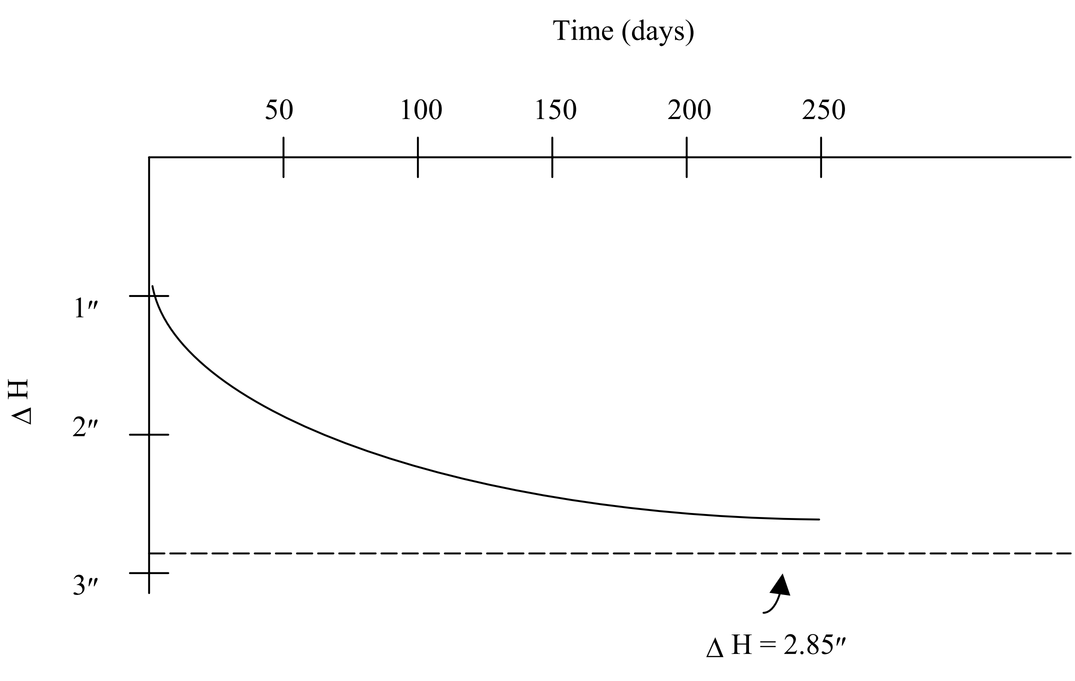
**Step 1: Obtain time for various settlement percentages.**

$t = \dfrac{T H_V^{\,2}}{C_V}$ (for Clay; Sand occurs immediately)

(Refer to Apple Freeway Design Example Chapter 6).

| % Consol. Clay | ΔH (inch) | T | $\dfrac{H_V^{\,2}}{C_V}$ | $t_{days}$ |
|---|---|---|---|---|
| 20 | 0.39 | 0.031 | 260 | 8 |
| 50 | 0.98 | 0.197 | | 51 |
| 70 | 1.37 | 0.408 | | 106 |
| 90 | 1.76 | 0.848 | | 220 |

**Step 2: Plot time-settlement curve for the pier.**



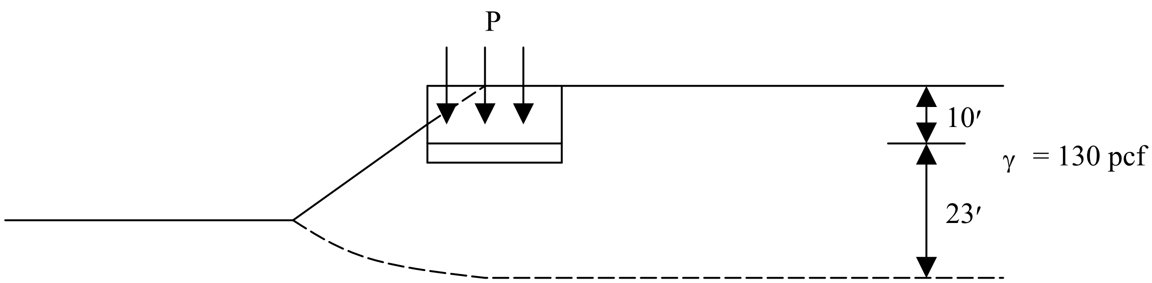

**East Abutment Footing Settlement.**

Assumptions:

- Abutment footing 10′ below top of fill
- Footing width = 7′

- Footing design pressure = 6300 psf (≈ 3 TSF)
- No internal embankment consolidation
- Organic layer excavated

**Compute abutment-footing settlement.**




**Step 1: Obtain net footing pressure.**

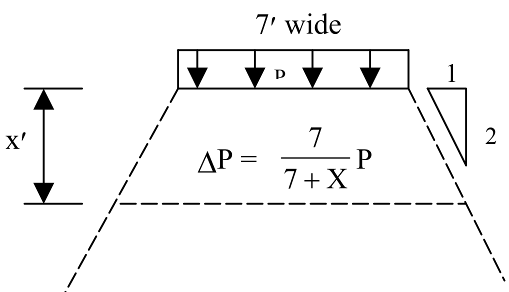
P = The net pressure applied at footing level assuming abutment constructed after embankment

P = 6300 psf – 1300 psf (Soil removed after waiting period)

P = 5000 psf

**Step 2: Determine pressure distribution.**

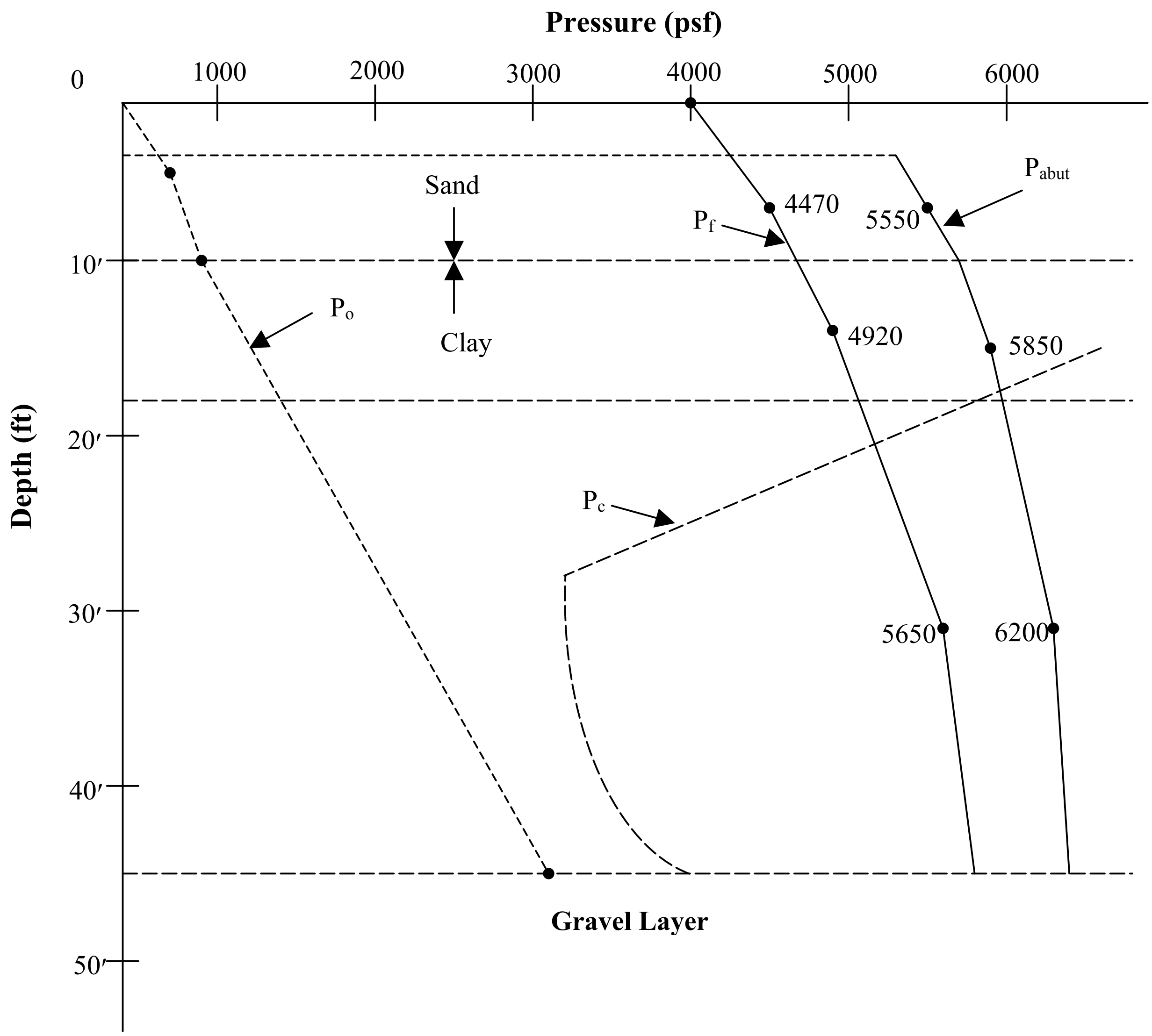
By 2 on 1 method – abutment




$$\Delta P = \left(\frac{7}{7+x}\right) 5000$$

| X Feet | ΔP psf |
|---|---|
| 23 (Ground Surface) | 1160 |
| 30 | 945 |
| 40 | 740 |
| 50 | 610 |
| 60 | 520 |

$P_F + \Delta P_{PSF} = P_{ABUT}$




**Step 3: Compute settlement in each layer.**

Abutment Settlement

- Layer 2 – Sand 3′ - 10′

$$\Delta H = H\frac{1}{C'}\mathrm{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 7\frac{1}{90}\text{Log}\frac{5550}{4470}$$

$\Delta H = 0.0065' \sim 0.08''$

- Layer 3 – Clay

10′ - 17′ (All Preconsolidated)

$$\Delta H = H\frac{C_R}{1+e_0}\text{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 7\frac{0.035}{1+0.97}\text{Log}\frac{5850}{4920}$$

$\Delta H = 0.009' \sim 0.11''$

17′ - 45′ (Not Preconsolidated)

$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_{ABUT}}{P_F}$$

$$\Delta H = 28\frac{0.35}{1+0.97}\text{Log}\frac{6200}{5650}$$

$\Delta H = 0.20' \sim 2.40''$

| Total East Abutment Settlement | |
|---|---|
| Layer 2 | 0.08 |
| Layer 3 | 0.11 |
| | 2.40 |
| $\Delta H_{ABUT} =$ | 2.59″ |

**Step 4: Determine time for settlement to occur.**

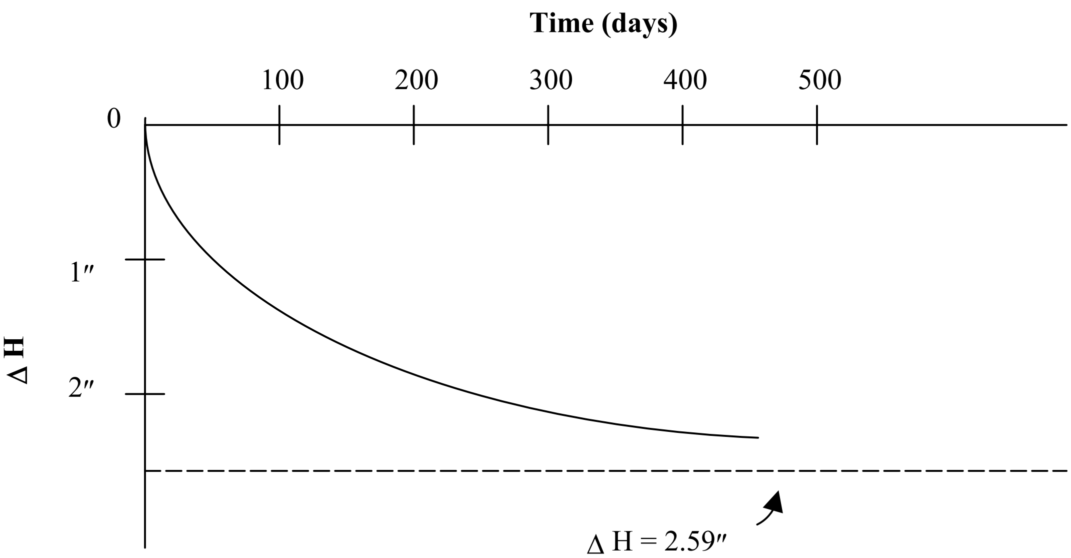
Time for settlement to occur

$$t = \frac{T H_V^2}{C_V}$$

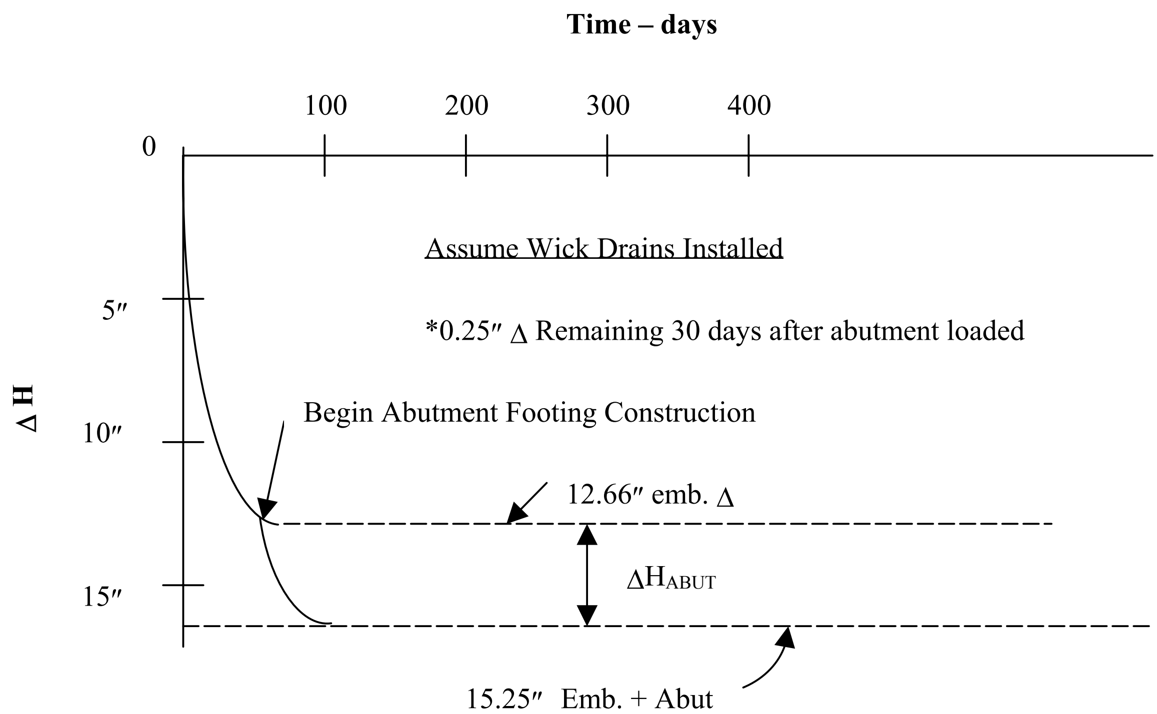
$H_V = 17.5'$

$C_V = 0.6\ ft^2/day$

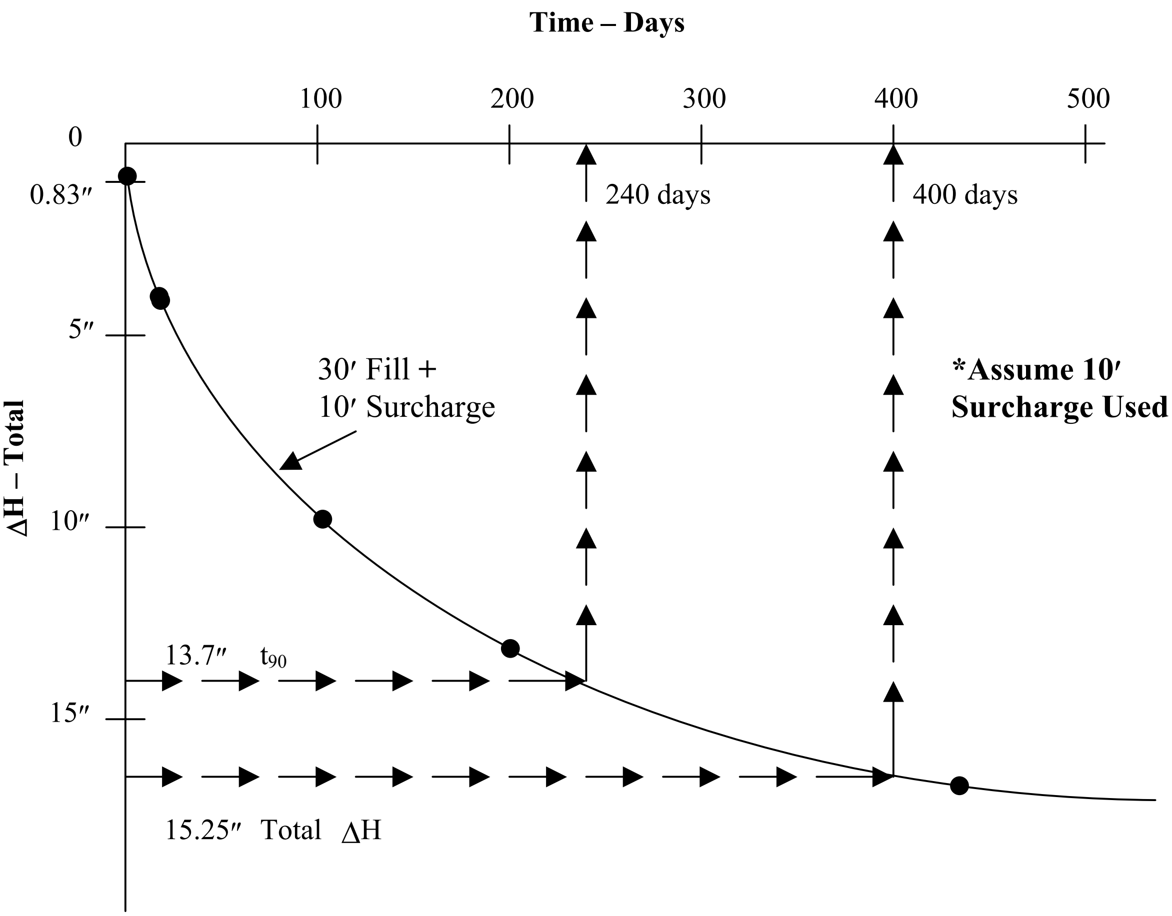
| % Consol. Layer 3 | $\Delta$H (inches) | T | $\frac{H_V^2}{C_V}$ | $t_{days}$ |
|---|---|---|---|---|
| 20 | 0.50 | 0.031 | 510.4 | 16 |
| 50 | 1.25 | 0.197 | | 101 |
| 70 | 1.76 | 0.403 | | 206 |
| 90 | 2.23 | 0.848 | | 433 |




Actual Abutment settlement will include settlement remaining due to embankment load. However surcharges and/or drains can be used to consolidate the clay layer for the embankments load and the abutment load.



Time – days
100
200
300
400
0
5″
10″
15″
Δ H
Assume Wick Drains Installed
*0.25″ Δ Remaining 30 days after abutment loaded
Begin Abutment Footing Construction
12.66″ emb. Δ
ΔHABUT
15.25″ Emb. + Abut




* Surcharge must be left in place for 13 months to dissipate all embankment and abutment ΔH

**Summary of the Spread Footing Design Phase for Apple Freeway Design Problem**

- Design Soil Profile

  Strength and consolidation values selected for all soil layers. Footing elevation and width chosen.

- Pier Bearing Capacity

  $Q_{allowable}$ = 3 tons/sq.ft.

- Pier Settlement

  Settlement = 2.8", $t_{90}$ = 220 days.

- Abutment Settlement

  Settlement - 2.6", $t_{90}$ = 433 days.

- Vertical Drains

  $t_{90}$ = 60 days - could reduce settlement to 0.25" after abutment constructed and loaded.

- Surcharge

  10' surcharge: $t_{90}$ = 240 days
  before abutment constructed.

# CHAPTER 8.0
# DEEP FOUNDATION DESIGN

Foundation design and construction involves assessment of factors related to engineering and economics. The selection of the most feasible foundation type requires consideration of both shallow and deep foundation types in relation to the characteristics and constraints of the project and site conditions. A cost evaluation is essential in the selection of the optimum foundation system.

Situations commonly exist where shallow foundations are inappropriate for support of structural elements. These situations may be related either to the presence of unsuitable soil layers in the subsurface profile, adverse hydraulic conditions, or tolerable movements of the structure. Deep foundations are designed to transfer load thru unsuitable subsurface layers to suitable bearing strata. Deep foundation types include several pile types (driven, non-driven, micropiles, etc.) and drilled shafts. The suitability of a deposit may depend on bearing capacity, settlement or scour considerations. Foundation engineers should interact with both structural and hydraulic engineers in the design of deep foundations. This manual will only present the basic concepts of deep foundation design that are similar for all deep foundation types. More advanced design and construction information on deep foundations can be found in other FHWA manuals and training courses related to specific deep foundation topics. This chapter will illustrate the use of those concepts for driven pile design and peripherally extend those concepts to drilled shafts.

## 8.1 DRIVEN PILE FOUNDATION DESIGN

For many years the use of a pile foundation has meant security to many designers. The temptation to use piles under every facility is great; detailing of plans is routine, quantity estimate is neat, and safe structural support apparently assured. Unfortunately, the rationale behind pile type selection, length, and allowable load, is usually based on peripheral factors such as local availability, outdated dynamic formulas or previous usage of certain pile types. Traditionally, the duty of determining if a pile is "good" has been passed on to the inspector in the field, who seldom has any training in foundation design or is given any pertinent information on which to base driven pile acceptability.

Foundation engineers agree that proper pile design is a most difficult task; requiring a combination of theoretical training, and experience in design and construction. Unlike other foundation types which may be installed to close tolerances in shallow excavations, piles are brutally forced far below ground surface with hammers that may generate stresses in excess of permitted static design levels. At present, a wide variety of piling is in use; each possessing inherent characteristics which affect determination of both capacity and driveability. Alternate pile types for a particular project can be evaluated by applying engineering judgment.

## 8.2 ALTERNATE PILE TYPE EVALUATION

There are many different methods of classifying piles such that several types may be suitable for a given situation. Some factors to be considered are as follows:

1. Pile Material

| | Optimum Load Range | Optimum Length |
|---|---|---|
| Timber | 30-80 Kips | 20-40 ft. |
| Concrete | 80-800 Kips | 40-150 ft. |
| Steel H | 80-400 Kips | 40-160 ft. |
| Steel Pipe | 70-1000 Kips | 30-100 ft. |

2. Pile Shape Effects

| | Pile Types | Effects |
|---|---|---|
| Displacement | Steel Pipe (Closed end), Concrete | Increase lateral ground stress |
| | | Densify cohesionless soils, remolds and weakens cohesive soils temporarily |
| | | Set-up time may be 6 months in clays for pile groups |
| Nondisplacement | Steel H, Steel Pipe (Open end) | Minimal disturbance to soil |
| | | Not suited for friction pile in granular soils |
| Tapered | Timber, Monotube | Increased densification of soils with less disturbance, high capacity for short length in granular soils |

3. Subsurface Conditions

| Typical Problem | Advice |
|---|---|
| Boulders overlying bearing stratum | Use heavy nondisplacement pile with a reinforced tip or manufactured point and include contingent predrilling item in contract. |
| Loose cohesionless soil | Use tapered pile to develop maximum skin friction. |
| Negative skin friction | Use smooth steel pile to minimize drag adhesion, and avoid battered piles. Provide bitumen coating in drag zone. |
| Deep soft clay | Use rough concrete pile to increase adhesion and rate of pore water dissipation. |
| Artesian Pressure | Do not use mandrel driven thin-wall shells as generated hydrostatic pressure may cause shell collapse; pile heave common to closed-end pipe. |
| Scour | Do not use tapered piles unless large part of taper extends well below scour depth. Design permanent pile capacity to mobilize soil resistance below scour depth. |
| Coarse Gravel Deposits | Use precast concrete piles where hard driving expected in coarse soils. DO NOT use H-piles as nondisplacement piles will penetrate at low blow count and cause unnecessary overruns. |

4. Location and Topography

Problems to Consider:

- Driven piles may cause vibration damage.
- Access to remote area may restrict driving equipment size and, therefore, pile size.
- Local availability of certain materials may have decisive effects on pile selection.
- Waterborne operations may dictate use of shorter pile sections due to pile handling limitations.
- Steep terrain may make the use of certain pile equipment costly or impossible.

5. Structural Characteristics of the Proposed Superstructure

- Heavy structures may require stiff high capacity piles for lateral load resistance.
- Small, isolated structures may dictate small piles as mobilization costs for large driving equipment may be excessive.

Frequently consideration of pile type for a project will be influenced by the possible use of one pile type on several structures or at all footings on a particular structure. Designers should begin the selection process by choosing the two pile types which in their opinion will provide a cost-effective foundation for the project. The next step is evaluation between the selected alternates by determining required lengths and capacities at representative foundation locations.

## 8.3 DRIVEN PILE CAPACITY - STATIC ANALYSIS

The experienced foundation engineer has the ability to review boring data and classify zones in the subsoil with regard to relative pile support capability. However, without a rational method of design, this information will lead to pile selections that are at best wasteful, or at worst, dangerous.

Once the allowable structural load has been determined for prospective pile alternates, the pile length required to support that load must be determined. For many years this length determination was considered part of the "art of foundation engineering." In recent years more rational analytic procedures have been developed. Static analyses provide a useful design tool to select the most economical pile alternates. The methods which follow are established procedures which accurately account for the variables in pile length determination. The "art" remains in selecting appropriate soil strength values for the conditions and ascertaining the effects of pile installation on these values. For the typical project two static analyses will be required; the first to determine the length required for permanent support of the structures, and second to determine the soil resistance to be overcome during driving to achieve the estimated length. It must be stressed that each new site represents a new problem with unique boundary conditions. Experience with similar sites should not replace but refine the rational analysis methods presented herein.

## 8.4 COMPUTATION OF PILE CAPACITY

The ultimate capacity ($Q_{ult}$) of all driven piles may be expressed in terms of skin resistance ($Q_s$) and point resistance ($Q_p$);

$$Q_{ult} = Q_s + Q_p$$

The value of both $Q_s$ and $Q_p$ are determined in each layer based on either frictional or cohesive behavior of the soil. The strength of frictional soils is based on friction angle. Cohesive soil strength is based on undrained shear strength. The pile capacity of cohesive soil layers should not be computed with both friction angle and cohesion values.

### 8.4.1 Soils with Frictional Strength

A. Skin Resistance

1. Approximate $Q_s$ for preliminary cost estimates (not recommended for design)
   $Q_s$ (tons) = 0.02 N' D $C_d$ (Reduce $Q_s$ by 1/2 for H-piles) (8-1)

   Where: D = pile length below ground
   $C_d$ = pile perimeter
   N′ = SPT value corrected for overburden pressure

2. Determining $Q_s$ for design (Nordlund's Method)

This method is based on correlation with actual pile load test results. The pile shape and material are important factors included in this method.

$$Q_s = \sum_{0}^{D} K_\delta \, C_F \, P_d \frac{\sin(\omega + \delta)}{\cos\omega} C_d \, \Delta d \qquad (8\text{-}2)$$

Which simplifies for non-tapered piles ($\omega = 0$) to the following:

$$Q_s = \sum_{0}^{D} K_\delta \, C_F \, P_d \sin\delta \, C_d \, \Delta d \qquad (8\text{-}3)$$

Where: $Q_s$ = Total skin friction capacity
$K_\delta$ = Dimensionless factor relating normal stress and effective overburden pressure
$P_d$ = Effective overburden pressure at the center of depth increment d
$\omega$ = Angle of pile taper measured from the vertical
$\delta$ = Friction angle on the surface of sliding
$C_d$ = Pile perimeter
d = Depth increment below ground surface
$C_F$ = Correction factor for $K_\delta$ when $\delta \neq \phi$ (soil friction angle)

To avoid numerical integration, computations may be performed for pile segments of constant diameter ($\omega = 0$) within soil layers of the same effective unit weight and friction angle. Then equation 8-3 becomes:

$$q_s = K_\delta \, C_F \, P_d \sin\delta \, C_d \, D \qquad (8\text{-}4)$$

Where within the segment selected:

$P_d$ = The average effective overburden pressure in segment D
$C_d$ = The average pile perimeter
D = The segment length

$q_s$ = The capacity of pile segment D (skin friction)

Equation 8-4 can be more easily understood if skin friction is related to the shear strength of granular soils, i.e., normal force times tangent of friction angle, N Tan $\phi$. In equation 8-4 the terms $K_\delta$ $C_F$ $P_d$ represent the normal force against the pile, Sin $\delta$ represents the coefficient of friction between the pile and soil, and $C_dD$ is the surface area in contact with the soil. In effect equation 8-4 is a summation of the N Tan $\phi$ shearing resistance against the sides of the pile.

3. Computational steps for non-tapered piles are as follows:

   a. Draw the existing effective overburden pressure ($P_o$) diagram.

   b. Choose a trial pile length.

   c. Subdivide the pile according to changes in the unit weight or soil friction angle ($\phi$).

   d. Compute the average volume per foot of each segment (V).

   e. Enter Figure 8-1 with that volume and the pile type to determine $\delta/\phi$ and compute $\delta$.

   f. Enter the appropriate chart(s) in Figures 8-2 to 8-5 to determine $K_\delta$ for $\phi$.

   g. If $\delta \neq \phi$, enter Figure 8-6 with $\phi$ and $\delta/\phi$ to determine a correction factor $C_F$ to be applied to $K_\delta$.

   h. Determine the average values of effective overburden pressure and pile perimeter for each pile segment.

   i. Compute $q_s$ from equation 8-4 for all pile segments and sum to find the ultimate frictional resistance developed by the pile.

For tapered piles Figures 8-2 to 8-5 must be entered with both $\varphi$ and $\omega$ to determine $K_\delta$. Also equation 8-2 should be used to compute the capacity of tapered piles. It is recommended that Nordlund's original paper in the May 1963 ASCE Journal (SMF) be referred to for numerical examples of tapered pile static analysis.

Selection of design friction angle should be done conservatively for piles embedded in coarse granular deposits. Pile load tests indicate that predicted skin friction is often overestimated; particularly in soil deposits containing either uniform sized or rounded particles. A conservative approach is to limit the shearing resistance by neglecting interlock forces. This results in maximum friction angle in predominately gravel deposits of $32^o$ for soft or rounded particles and $36^o$ for hard angular deposits. The Nordlund method also tends to overpredict capacity for piles larger than 24 inches in nominal width.

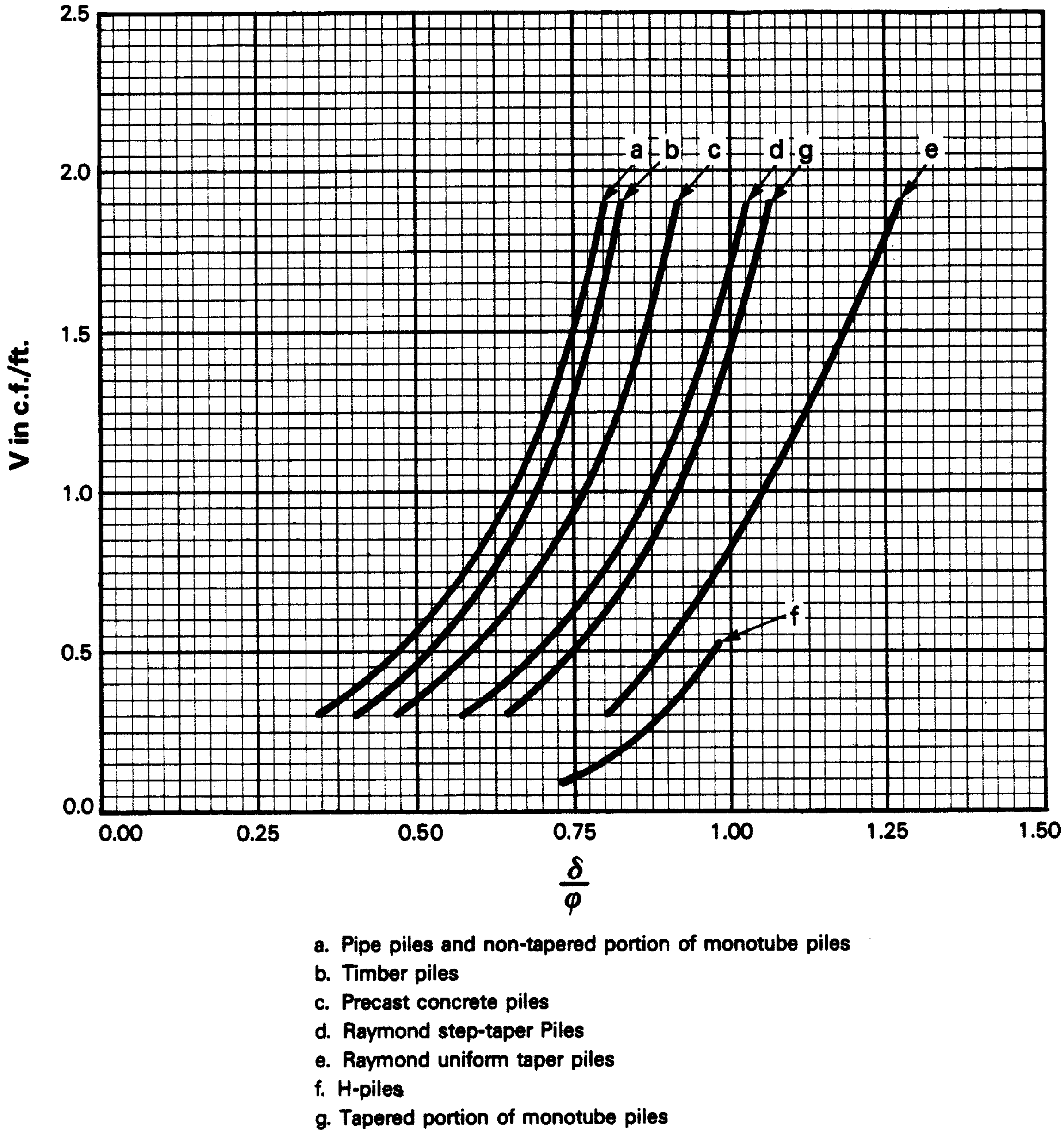



Figure 8-1: Relation of δ/ϕ and pile displacement, V, for various types of piles

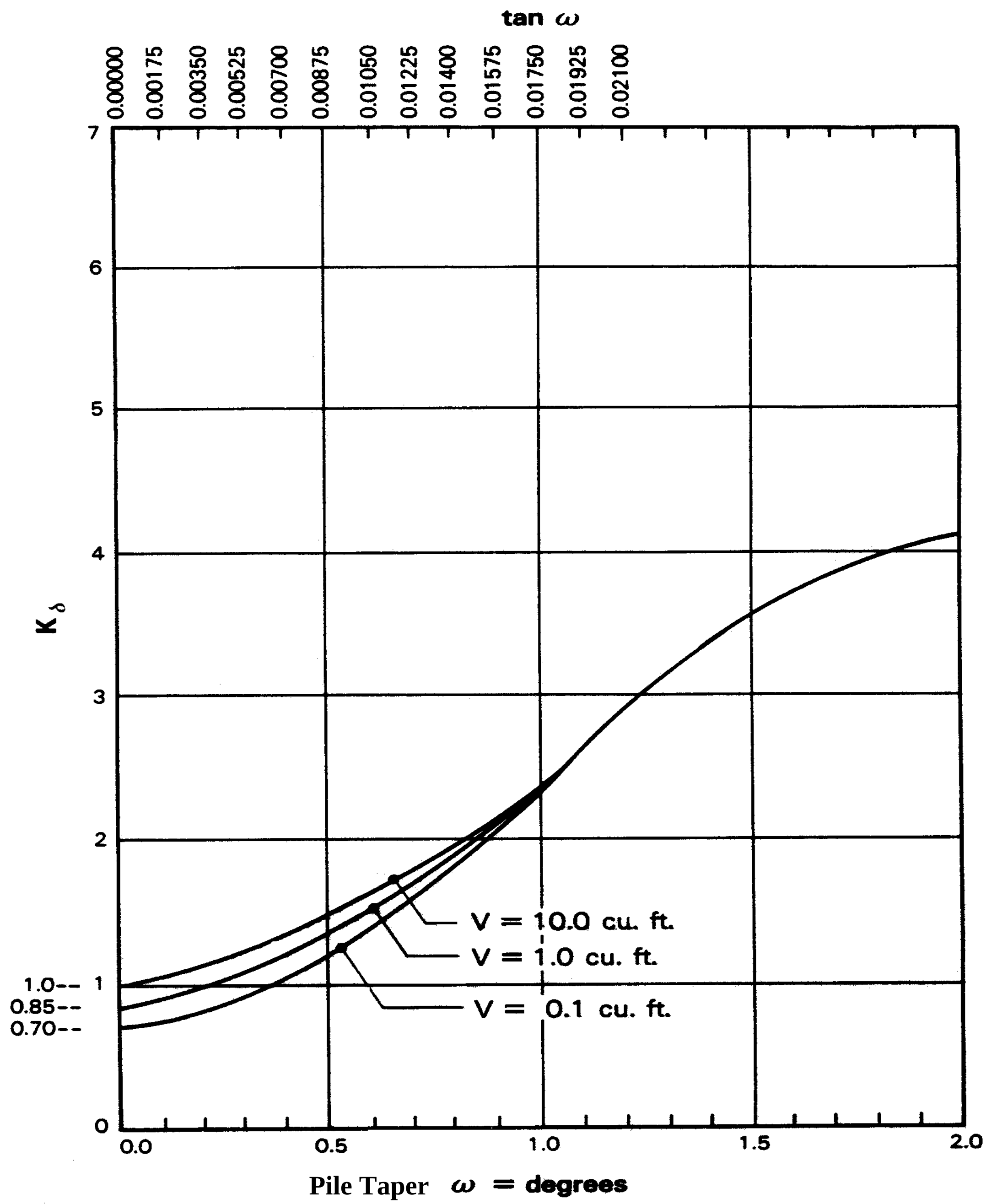



Figure 8-2: Design curves for evaluating $K_\delta$ for piles when $\phi = 25°$ (After Nordlund 1979)

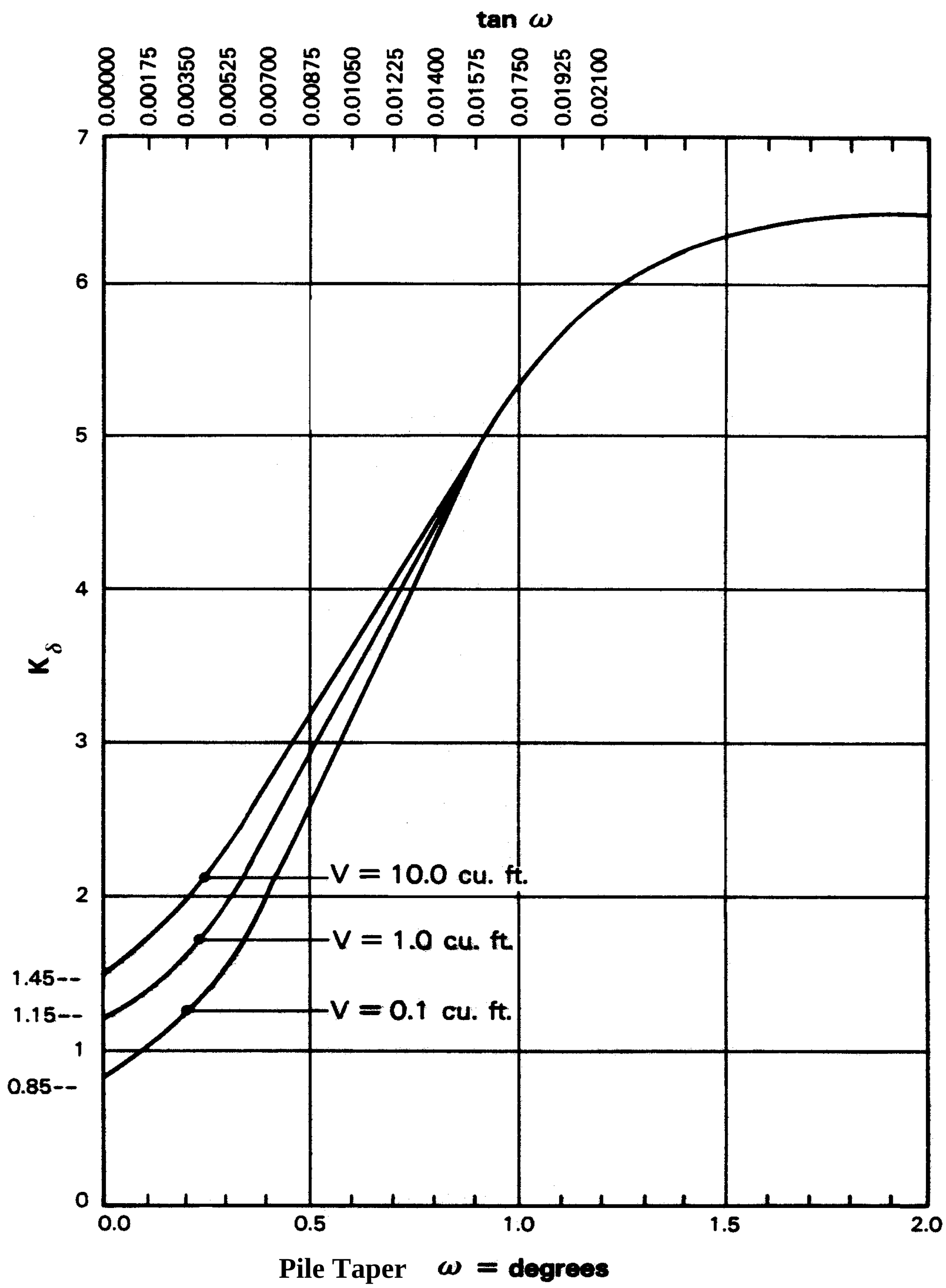



Figure 8-3: Design curves for evaluating $K_\delta$ for piles when $\phi = 30°$ (After Nordlund 1979)

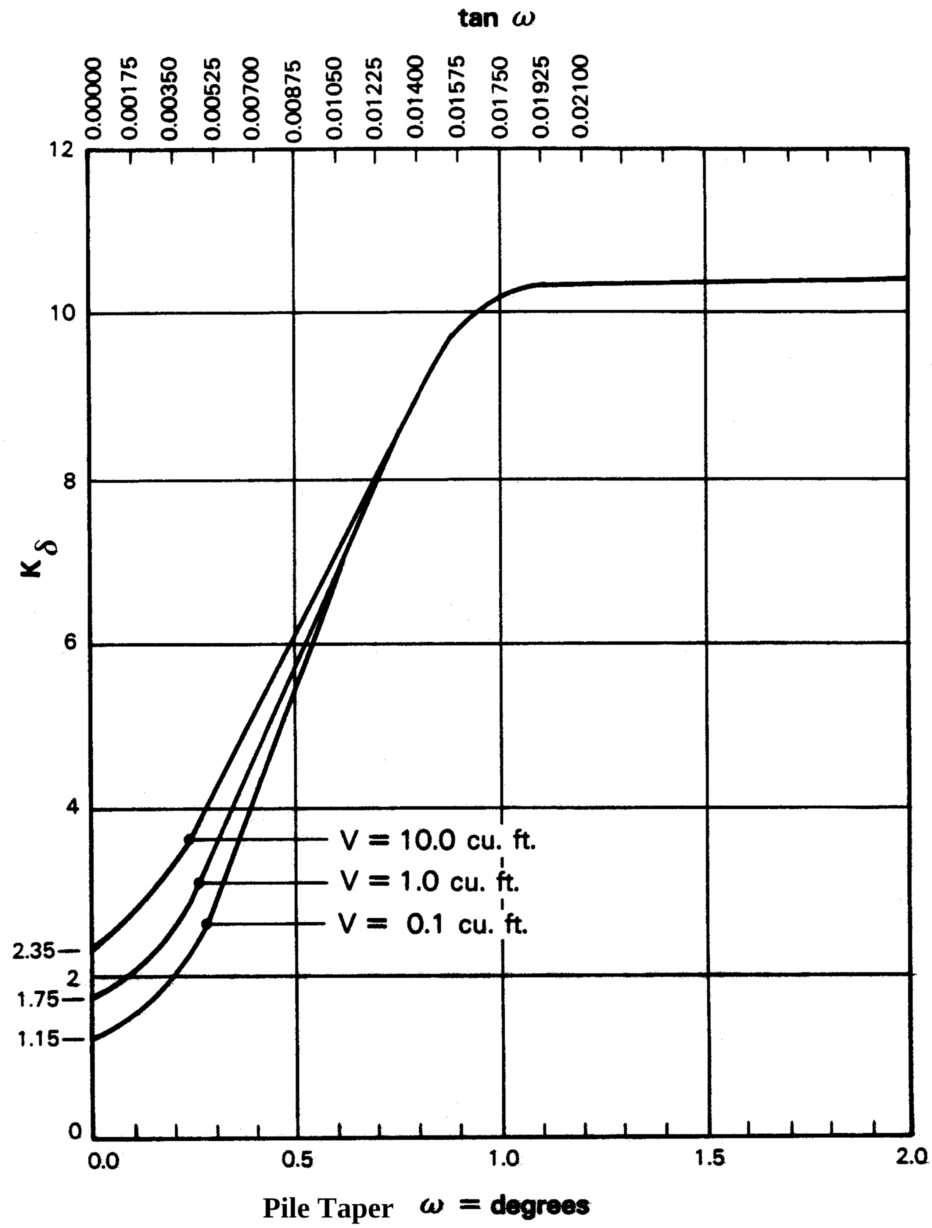



Figure 8-4: Design curves for evaluating $K_{\delta}$ for piles when $\phi = 35°$ (After Nordlund 1979)

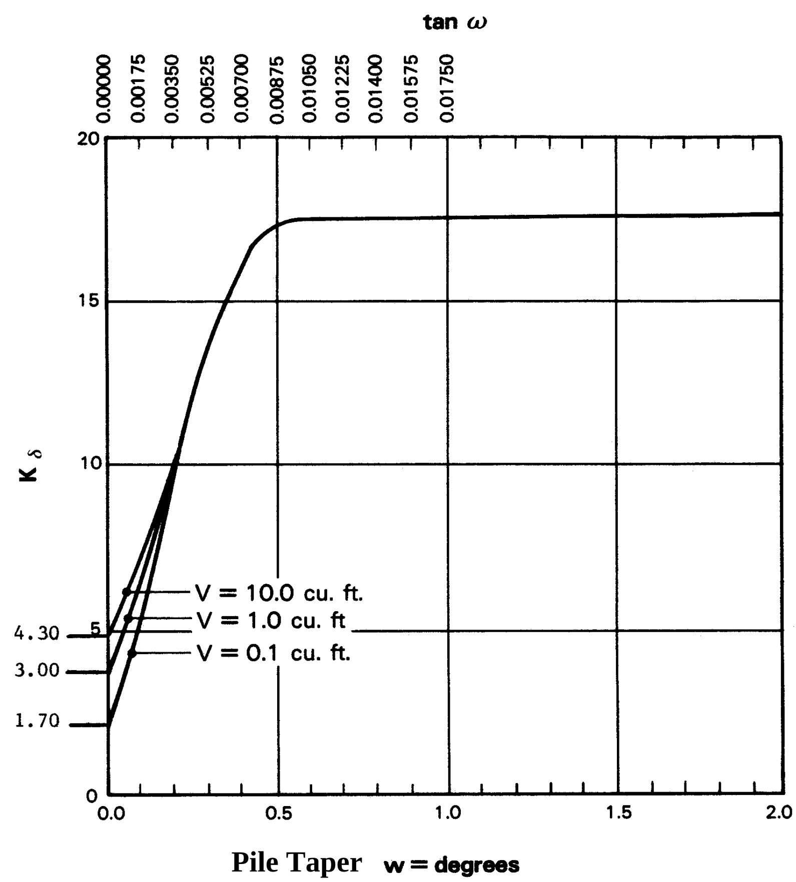



Figure 8-5: Design curves for evaluating $K_\delta$ for piles when $\phi = 40°$ (After Nordlund 1979)

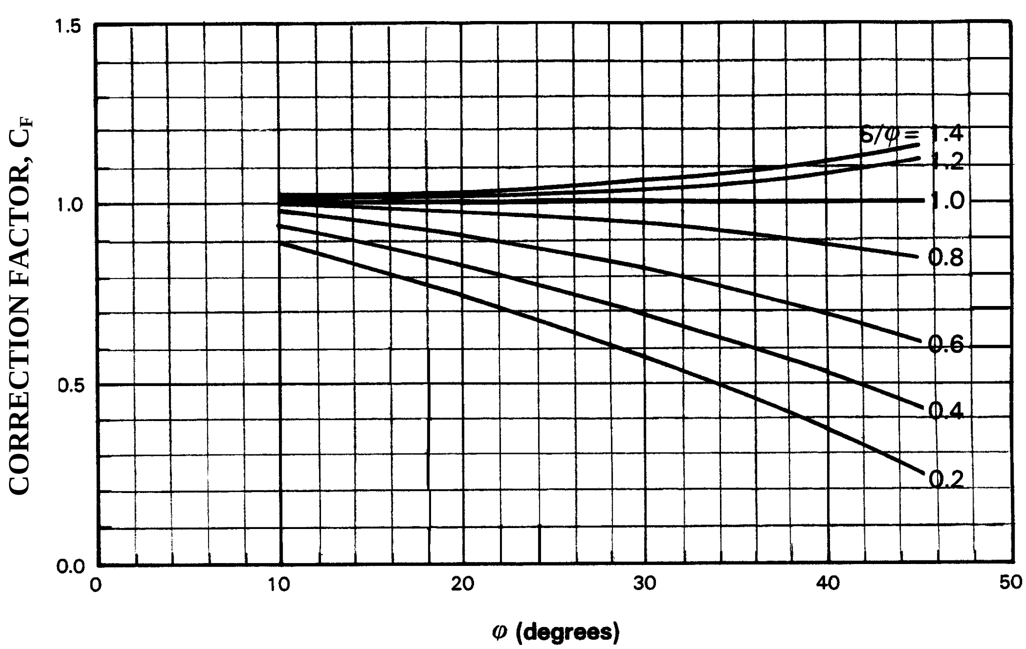



Figure 8-6: Correction factor, $C_F$ for $K_\delta$ when $\delta \neq \phi$

B. Point Resistance

1. Approximate $Q_p$ for preliminary cost estimate (not recommended for design).

$$Q_p \text{ (tons)} = 4\, N'\, A_p \tag{8-5}$$

Where: $A_p$ = area of pile point
$N'$ = SPT value corrected for overburden pressure

2. Determine $Q_p$ for design (Thurman's Method)

$$Q_p = A_p\, \alpha\, P_D\, N'_q \tag{8-6}$$

Where: $Q_p$ = end bearing capacity
$A_p$ = pile end area
$\alpha$ = dimensionless factor dependent on depth-width relationship (see Figure 8-7A)
$P_D$ = effective overburden pressure at the pile point
$N'_q$ = bearing capacity factor from Figure 8-7A

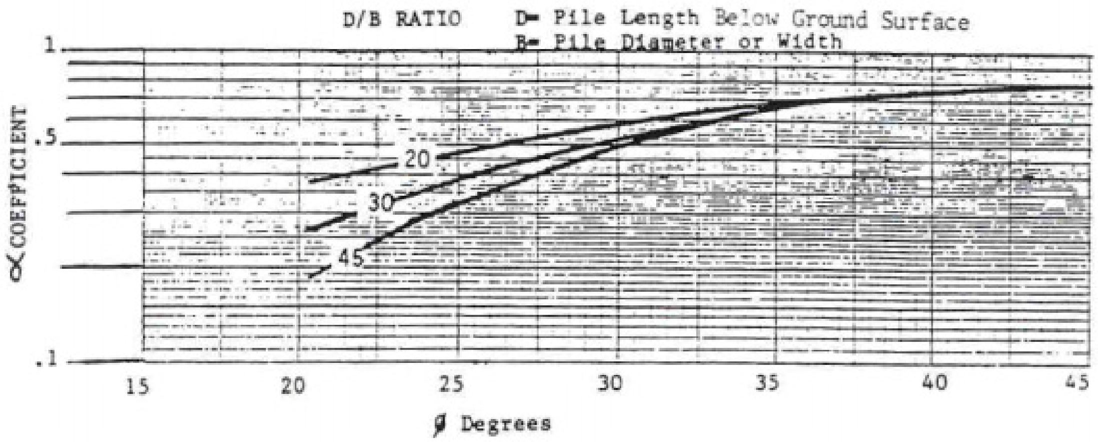
The $Q_p$ value is limited due to soil arching, which occurs around the pile point as the depth of tip embedment increases. For this reason, Nordlund has suggested limiting the overburden pressure at the pile point, $P_D$, to 3000 psf. More recently, other authors have suggested that further limitation must be placed on the end bearing so as not to compute unrealistic values. **Therefore, the $Q_p$ value computed from the equation should be checked against the limiting value, $Q_{LIM}$, obtained from the product of the pile end area and the limiting point resistance in Figure 8-7B. The end bearing capacity should be taken as the lesser of $Q_p$ or $Q_{LIM}$.**

The following criteria are suggested to determine the proper cross sectional end area to be used for H-piles.

a. Use the actual steel area if the point is founded on rock or in soil deposits having GRAVEL as the major component. Reinforced tips can be used to increase point area.

b. Use the enclosed area for all other soil deposits.

c. The ultimate pile capacity is the sum of all $q_s$ values and $Q_p$ which are below the deepest soil layer not considered suitable to permanently support the pile foundation. For scour piles, only sum $Q_p$ and those $q_s$ values below the anticipated scour depth.



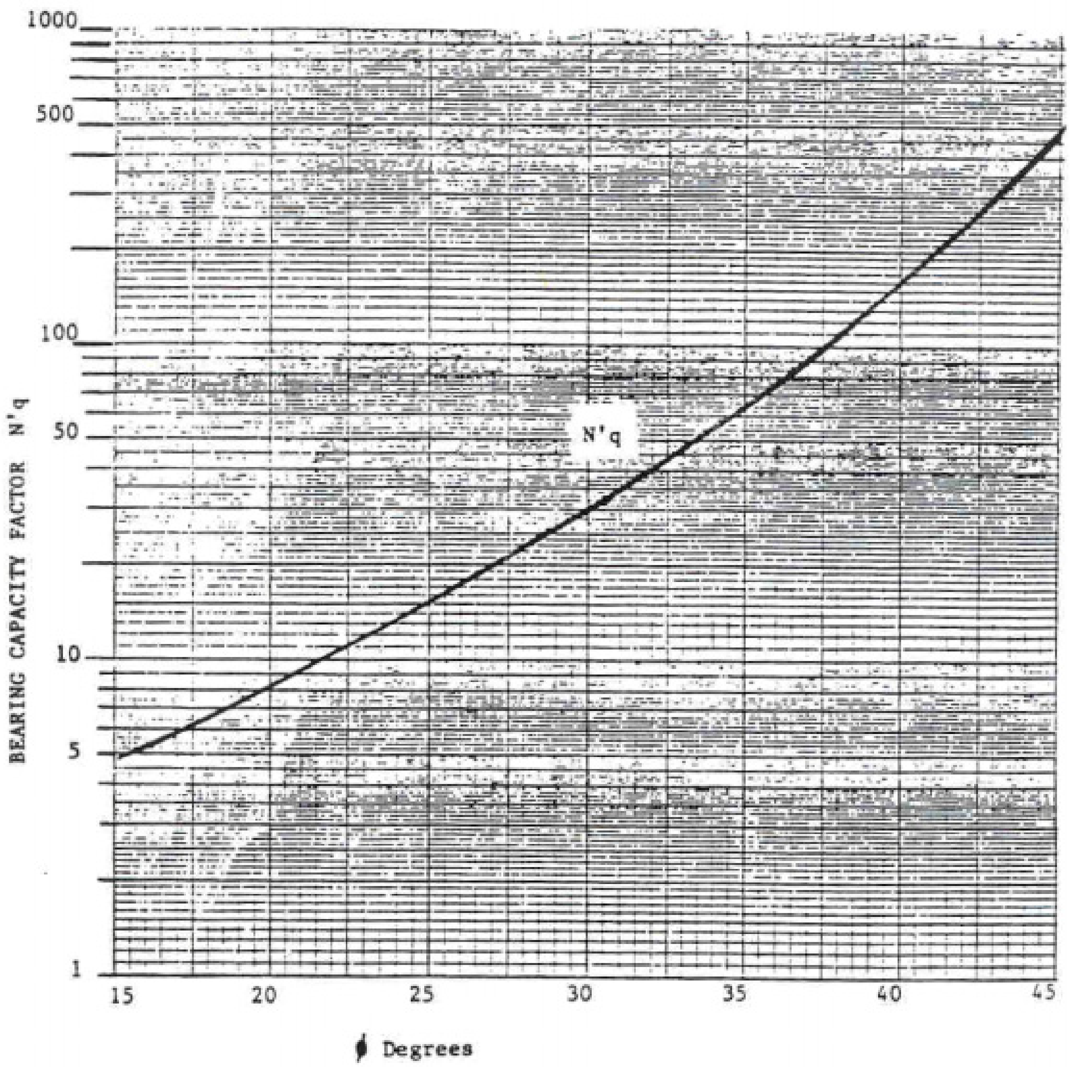




Figure 8-7A: Determination of ∀ coefficient and variation of bearing capacity factors with N

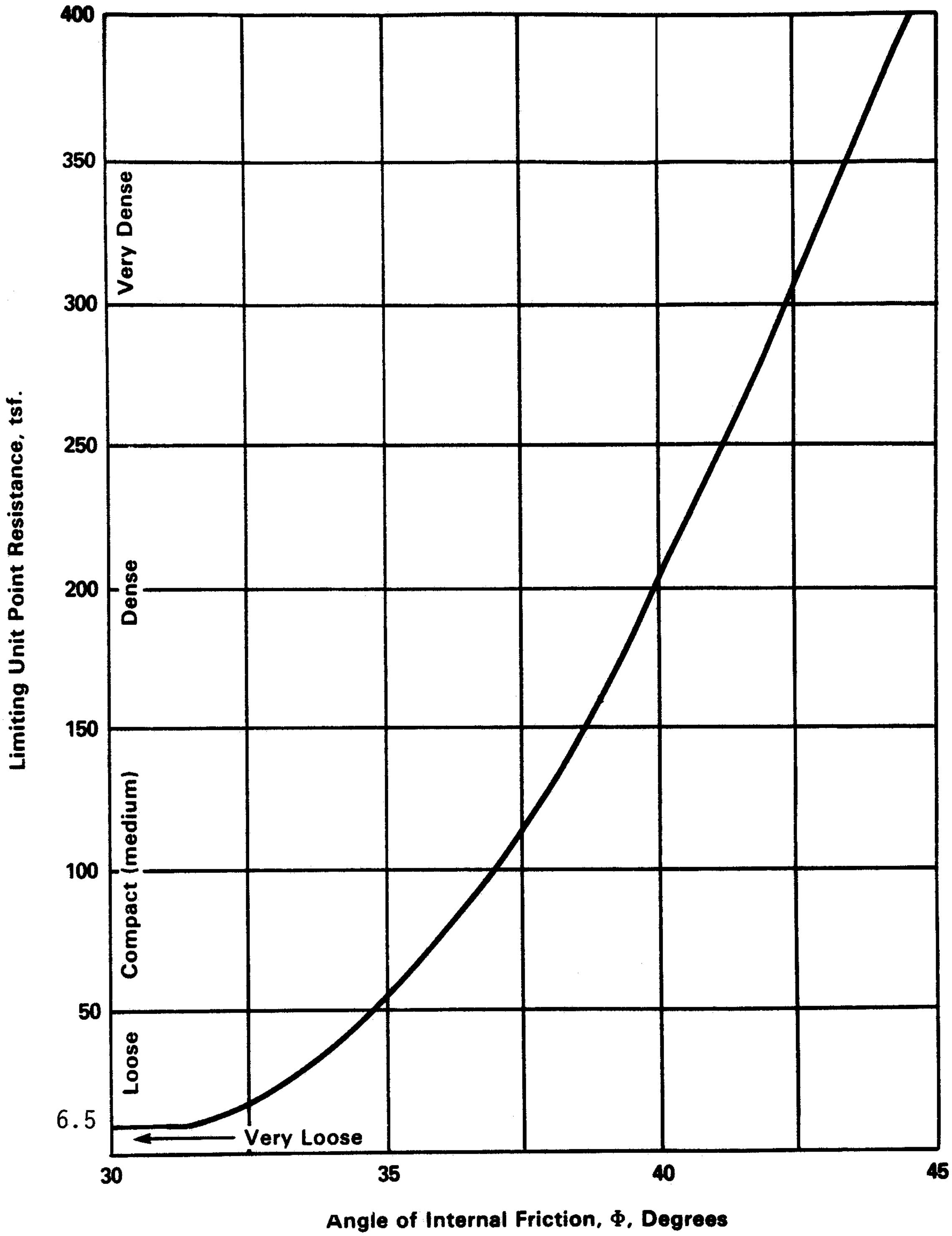
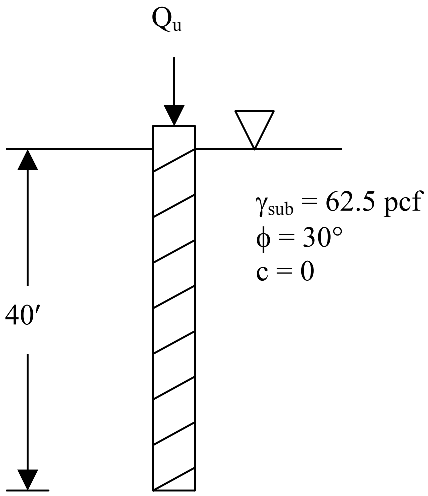



Figure 8-7B: Relationship between maximum unit pile point resistance and friction angle for cohesionless soils (After Meyerhof, 1976)

**Example 8-1:** Determine The Ultimate Capacity, $Q_u$, For The 1' Square Precast Concrete Pile




$Q_u = A_p\ \alpha\ P_D\ N'_q + K_\delta\ C_F\ P_d\ \mathrm{Sin}\ (\delta + \omega)\ C_d\ D$

Where the following terms are known from the problem

$A_p = 1$ sq.ft
$P_D = 40\ \gamma_{sub} = 2500$ psf
$P_d = 20\ \gamma_{sub} = 1250$ psf
$\omega = 0°$, $D = 40'$, $C_d = 4'$

Solution:

Find Point Resistance, $Q_P$:

Use Figure 8-7A to Find $N'_q$ and $\alpha$ for $\phi = 30°$

$N'_q = 30$ $\qquad\qquad \alpha = 0.5$ (for $\frac{D}{B} = 40$)

$Q_p = A_p \alpha P_D N'_q$

$= (1\text{ sq.ft})(0.5)(2500\text{ psf})\ 30 = 18.75$ tons

Check Limiting Point Resistance from Figure 8-7B

$Q_{Lim} = Q_{Lim}\ A_p = (6.5\text{ tsf})(1\text{ sq.ft}) = 6.5\text{ tons} \therefore Q_p = 6.5$ tons

Find Skin Resistance, $Q_S$: Use Figures 8-1, 8-3, and 8-6 with $\phi = 30°$

Figure 8-1 – For V = 1 cubic ft. per ft., and curve "C" for precast concrete piles;

$\frac{\delta}{\phi} = 0.76, \qquad \text{Since } \phi = 30°, \quad \delta = 22.8°$

Fig. 8-3 – For $\omega = 0$, V = 1 cu.ft/ft ;

$K_\delta = 1.15$

Fig. 8-6 – For $\frac{\delta}{\phi} = 0.76$;

$C_F = 0.9$

$Q_s = K_\delta\, C_F\, P_d \text{ Sin } \delta\, C_d\, D$

$Q_s$ = (1.15)(0.9)(1250 psf)(Sin 22.8)(4') 40'
$Q_s$ = 40.1 tons

$Q_{ult}$ = 6.5 + 40.1 = 46.6 tons

### 8.4.2 Soils with Cohesive Strength

The method of installation has considerable effect on the capacity of a pile driven into cohesive soil. The following facts have been established:

A. Pile capacity immediately after driving is the lowest mobilized during its useful life. This capacity may be estimated from the remolded vane shear strength or from soil sensitivity.

B. Pile capacity begins increasing immediately after driving ceases. The rate of increase in capacity depends on:

1. Pile size and pile spacing.
2. Pile type.
3. Hammer size
4. Drainage characteristics of foundation soil.

Accurate estimates of rate of increase in capacity can only be found by installing piezometers within the group and monitoring the rate of pore pressure decrease. In general, piezometers should be installed within 3 pile diameters of the pile. The rate of pore pressure decrease can be estimated from procedures shown in geotechnical publications such as ASCE Proceedings, November 1979, Ismael & Klym, "Pore Pressures Induced by Pile Driving".

C. Except for low strength cohesive soils, the long-term strength of the soil supporting the piles will be less than the original soil strength due to remolding during pile installation. Soils with higher in situ strengths will exhibit a greater percentage reduction in strength available for long-term pile support. Therefore, a reduction factor should be applied to undrained shear strengths used in static analysis computations for pile side friction. Figure 8-8 shows the reduced pile adhesion values for corresponding undrained shear strength values. The ultimate bearing capacity of a pile in cohesive soil is:

$$Q_{ult} = Q_s + Q_p$$

$$Q_{ult} = C_a\, C_d\, D + 9\, C_u\, A_p \qquad (8\text{-}7)$$

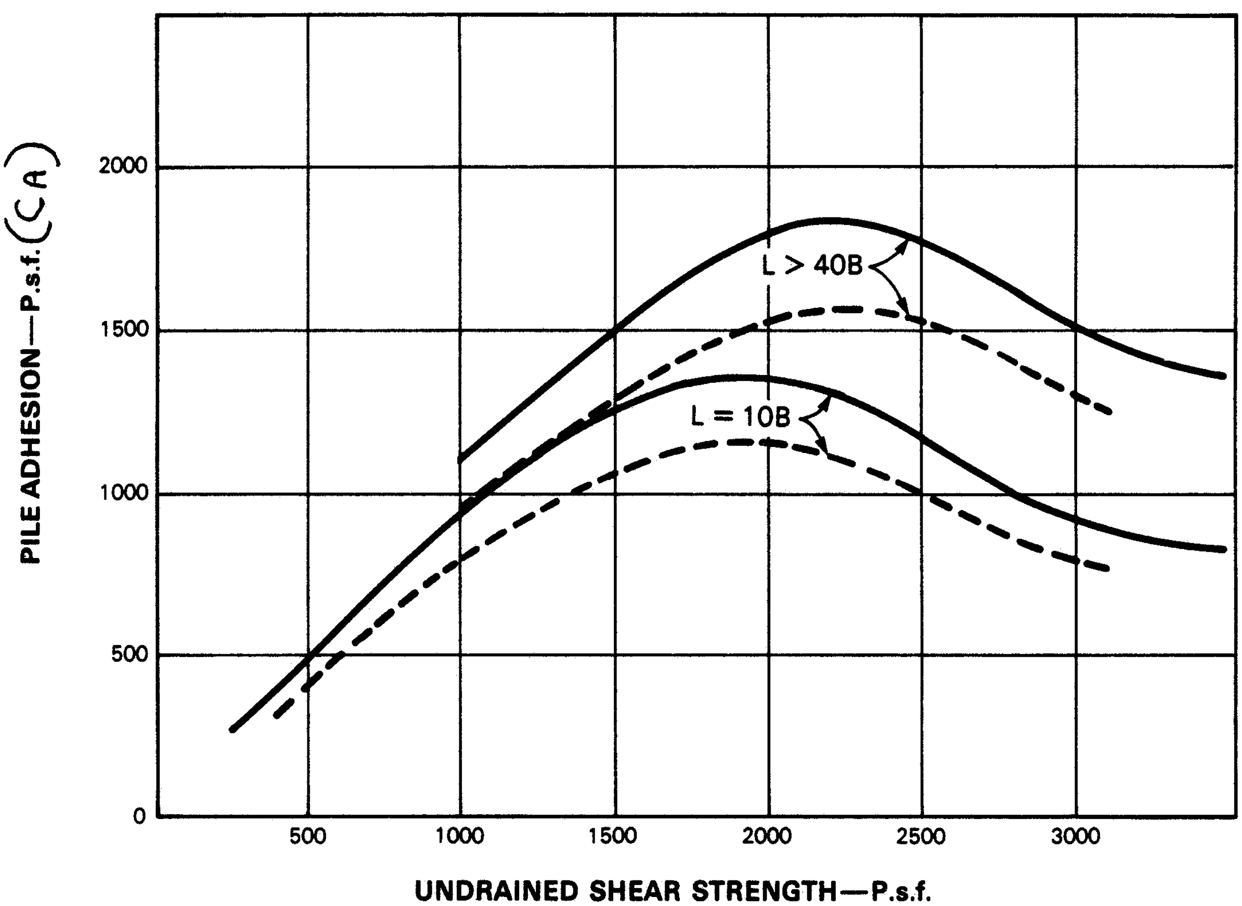
Where: $C_a$ = pile adhesion from Figure 8-8
$C_d$ = perimeter of pile
$D$ = pile segment length
$C_u$ = undrained shear strength

$A_p$ = pile end area

However, the $Q_p$ value of 9 $C_u$ $A_p$ is usually taken as zero because substantial movement of the pile tip (~1/10 of the pile diameter) is needed to mobilize end bearing capacity.

In the case of H-piles, calculate:

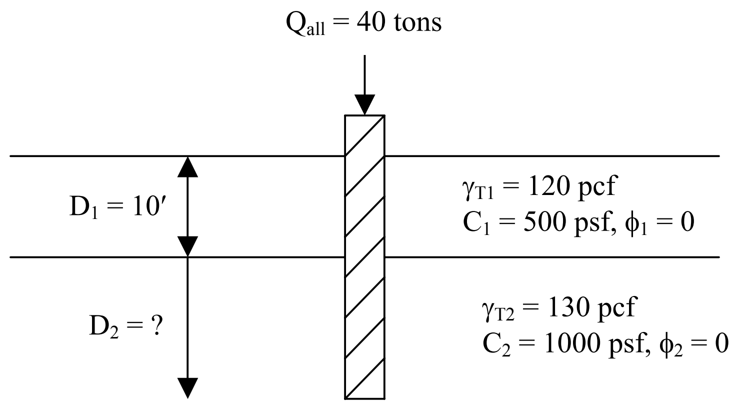
1. The perimeter ($C_d$) as twice the sum of the widths of the flange and web.
2. The pile end area ($A_p$) as the product of the flange and web dimensions.
3. The adhesion ($C_a$) from the curves for corrugated piles in Figure 8-8 as the actual value involves two pile faces of smooth steel adhesion and two faces where pure soil shear occurs.




Legend: L = Distance from ground surface to bottom of clay layer or pile tip; whichever is less
B = Pile diameter
Concrete, timber, corrugated steel piles
Smooth steel piles

Figure 8-8: Adhesion values for piles in cohesive soils

**Example 8-2:** Determine the Required Pile Length To Resist A 40 Tons Load with A Safety Factor Of 2. Assume No Point Capacity For the 1′ Square Precast Concrete Pile.




Solution:

$Q_u = C_{a1}\, C_{d1}\, D_1 + C_{a2}\, C_{d2}\, D_2$

$C_{d1} = C_{d2} = 4 \times 1' = 4'$

From Figure 8-8

$C_{a1}$ = 500 psf
$C_{a2}$ = 1100 psf

$Q_u$ = 40 tons × 2 = 80 tons = (500 psf)(4′)(10′) + (1100 psf)(4′)$D_2$

$$D_2 = \frac{80-10}{2.2} \approx 32'$$

$\therefore$ Total pile length required = 32′ + 10′ ≈ 42′

## 8.5 PRACTICAL ASPECTS OF DRIVEN PILE DESIGN

Prediction of pile capacity is a powerful tool for the designer. Cost-effective pile types can be selected and pile lengths established in design with confidence, if common sense is used in applying the calculations.

First, the ultimate capacity, soil resistance contributing toward non-yielding support of the pile load, should be only considered below any unsuitable, compressible soil layers. The main reason piles are used is to transfer the structure load through poor soils to competent soils. Therefore, the resistance obtained in or above these layers should not contribute toward the required design load. For example, a pile to be driven through a dense sand layer overlying a soft clay layer and finally a deep gravel layer, should be designed to mobilize all necessary support capacity only in the gravel layer. Similarly, scour piles can only mobilize useful resistance below the expected scour depth.
Second, the driving capacity, soil resistance to be overcome to drive the pile to the required length, must be computed. This value is not the same as the design load which contains a safety factor and may

disregard soil resistance in overlying layers. This resistance may easily be obtained after the design pile length is established by adding up the static analysis values for all soil layers with consideration of soil strength loss due to driving in fine-grained soil layers. The driving capacity in fine-grained soil layers can be computed by dividing the static capacity of the soil layer by the soil sensitivity,ie., the ratio of undisturbed strength to the remolded strength. A special driving capacity case is the restrike capacity. Restrike capacity refers to the condition when all layers in the soil profile have had sufficient time to consolidate (setup) around the pile. No safety factor is applied to driving capacity because the actual soil resistance must be known by the designer to establish pile section or thickness, and for the contractor to estimate how big a hammer to use. In example 8-3, the static capacities computed in both the sand layer and gravel layer would be added to the capacity in the clay layer divided by the clay sensitivity.

Third, the designer should carefully choose the design safety factor used to establish the pile length required for a desired design load. The higher the safety factor in design, the greater the problem of pile installation in construction and the greater the need to perform wave equation analysis in design.

The safety factor selected should be based on both the quality of the subsurface information and the degree of construction control to be used for production pile driving. Assuming that procedures recommended in this manual are used for foundation investigation, the following safety factors on ultimate static capacity of piles supported by soil should be employed based on pile construction control method to be used. Piles supported on rock with RQD > 50 percent may use safety factor of 2.0 assuming driveability has been confirmed by wave equation analysis.

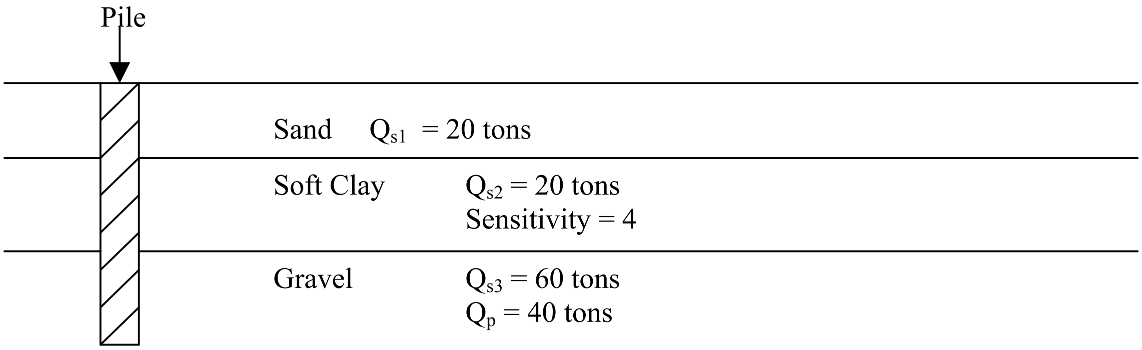
| **Control Method** | **Safety Factor** |
|---|---|
| Static Load Test and Wave Equation | 2.0 |
| Dynamic Load Test and Wave Equation | 2.25 |
| Test Piles and Wave Equation | 2.50 |
| Wave Equation | 2.75 |

Dynamic formula safety factors will be addressed in the construction control section.

### 8.5.1 Static Analysis Computer Programs

The user-friendly program SPILE was developed by FHWA in 1993 to permit rapid evaluation of the static capacity of alternate pile types. The user's manual is entitled "SPILE: Ultimate Static Capacity for Driven Piles" and is numbered FHWA TA-91-045. In 1998, a new Windows-based pile capacity program, DRIVEN, was developed to expand the capabilities of the SPILE program. The DRIVEN program program permits the user to enter the entire soil profile at a project. Based on this input, DRIVEN will calculate and plot pile capacities at predetermined intervals of depth for the profile depth. In addition the new analysis options featured in DRIVEN include multiple water tables, soft layer effects, scour effects, and open-end pipe pile design. Output options include ultimate capacity, driving capacity, and restrike capacity as well as an option to create a driveability file for subsequent wave equation analysis. The restrike capacity, which includes the setup capacity of all soil layers, is useful for evaluating the results of testing to verify post driving pile capacity.

**Example 8-3:** Find The Ultimate Capacity, The Driving Capacity And The Restrike Capacity For The Pile From The Static Capacity And Soil Values Listed In The Profile.




Solution:

Ultimate capacity = $Q_{s3} + Q_P = 60 + 40 = 100$ tons

Driving capacity = $Q_{s1} + (Q_{s2}/\text{Sensitivity}) + Q_{s3} + Q_P = 20 + \frac{20}{4} + 60 + 40 = 125$ tons

Restrike capacity = $Q_{s1} + Q_{s2} + Q_{s3} + Q_P = 20 + 20 + 60 + 40 = 140$ tons

## 8.6 DESIGN OF PILES FOR GROUP EFFECTS ON CAPACITY

The installation of a single pile results in disturbance in the surrounding soil that can extend outward a radial distance of six pile diameters depending on soil conditions and pile type. This disturbance can affect the capacity of piles within the group to carry load. The change in pile capacity due to group effects is commonly referred to as the efficiency of the pile group, E. The equation to determine the group capacity is:

$$P_{ult} = n\,E\,Q_{ult}$$

Where:

$P_{ult}$ = group capacity
$n$ = number of group piles
$E$ = group efficiency, and
$Q_{ult}$ = capacity of a single pile

### Granular Soils

In the case of granular soil, the disturbance generally causes additional compaction that can result in increased capacity for piles in the group. However in difficult driving situations, contractors may use predrilling or jetting methods that reduce long-term capacity. Also battered piles are less effected by densification than non-battered piles.

Piles that are spaced at least 3 diameters center-to-center in granular soils general act as individual piles. Assuming no predrilling or jetting, the group capacity may be conservatively calculated as:

$$P_{ult} = n\, Q_{ult}$$

**Cohesive Soils**

Studies have shown fine-grained soils immediately adjacent to the driven pile are completely disturbed with lesser amount of disturbance extending beyond four pile diameters of the pile face. The magnitude of this disturbance is increased in pile groups as adjacent piles are driven. Driving solid cross section piles, such as closed end pipes, causes more disturbance than open-end pipe piles. Jetting or predrilling oversized holes with "mud" can also affect the soil adjacent to the pile and complicate load transfer. However, the time for consolidation of this disturbed zone is governed by the same general principle as previously used for spread footings except horizontal drainage is of primary importance.

$$t = \frac{TH^2}{C_h} \tag{6-4}$$

Where: $C_h$ = The horizontal coefficient of consolidation.
$T$ = The time factor

For single piles, H is usually one or two pile diameters. For closely spaced groups, H may be the distance to the group exterior or the maximum vertical drainage path. Consolidation is also aided by escape of water along the pile face, particularly if the pile material is timber or concrete. Practically, consolidation of this zone occurs on most projects before the contractor is ready to build the superstructure.

In clays, the long-term capacity of the piles in the group may also be affected depending on the spacing of the piles and whether the pile cap is in firm contact with the ground. In general the center-to-center spacing of piles in a group should not be less than 3 diameters. The issues of pile cap contact, pile batter, and special installation effects on capacity are beyond the scope of this manual. Conservative group capacity design can be achieved in cohesive soils as follows depending on pile center-to center spacing:

Spacing 3 diameters to 6 diameters; $P_{ult} = 0.7\, n\, Q_{ult}$
Spacing greater than 6 diameters; $P_{ult} = n\, Q_{ult}$

## 8.7 DESIGN OF PILES FOR LATERAL LOAD

The theory and design method for analyzing laterally loaded piles is beyond the scope of this basic manual. Guidance on lateral load analysis is provided in FHWA-IP-84-11 "Handbook on Design of Piles and Drilled Shafts under Lateral Load." FHWA also funded the development of a user-friendly computer program, COM624P, for lateral load analysis. The COM624P user's manual version 2.0 is numbered FHWA-TA-91-048.

## 8.8 SETTLEMENT OF PILE FOUNDATIONS

The analysis for settlement of pile foundations resembles that for spread footings in that both are based on those principles which govern soil consolidation. The major differences which must be considered in determining pile settlement magnitude and time for occurrence follow.

### 8.8.1 Transfer of load to soil

The load applied to a spread footing foundation is transmitted to the soil only from the bottom of the footing. The formula shown below easily can be used for spread footings as all terms in the equation are defined with respect to depth.

$$\Delta H = H \frac{C_c}{1+e_0} Log \frac{P_F}{P_0} \tag{6-2a}$$

However, loads applied to pile foundations are transferred to the soil over the entire length of the pile as skin and point resistance. Referring to the formula for ΔH, neither the H nor $P_F$ terms in the equation are well defined as varying proportions of the pile load are transferred to the soil at various depths depending on the consolidation properties of the soil. Fortunately, a simple method has been developed to approximate settlements due to subsoil consolidation for basic load transfer situations. This method, which is shown in Figure 8-9, is suitable for pile foundations installed in groups. Practically, pile groups present the greatest possibility of settlement as the total load applied to a group is transmitted deep below the pile tips whereas single piles transfer load in the immediate vicinity of the pile.

### 8.8.2 Effects of installation

Spread footings are placed in carefully prepared excavations where every effort is made not to disturb the foundation soil. Consolidation properties may be assumed to be as found from lab testing of undisturbed samples. Piles are most commonly installed by brutally forcing the pile below ground with a large hammer.

Long after installation, driven piles can retain large residual stresses which significantly influence load-settlement characteristics. Particularly noticeable in granular soils, these stresses tend to reduce observed settlements. Actual pile settlements in granular soils are generally negligible.

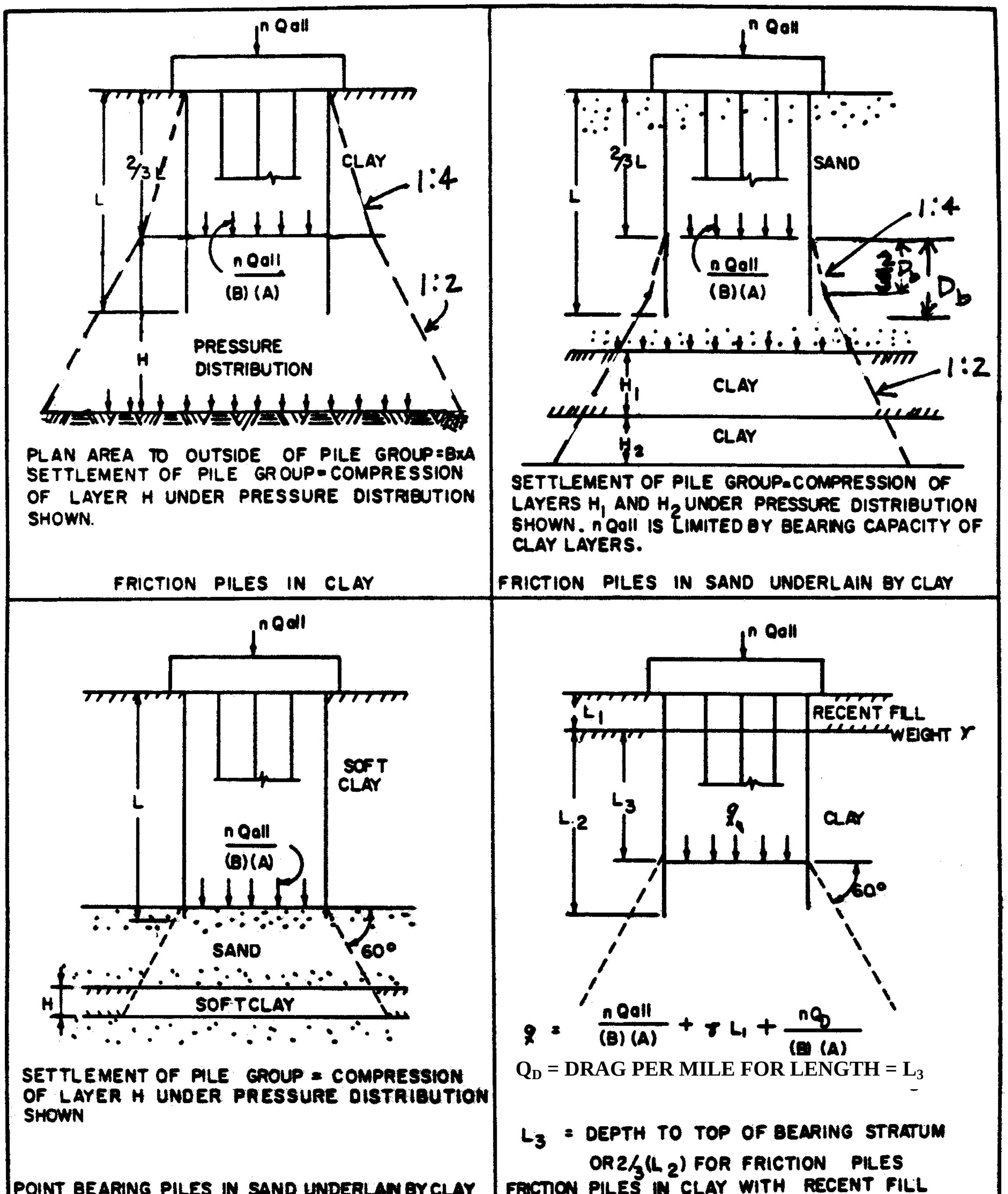
Practically, the foundation engineer should only be concerned about settlement of friction piles which terminate in cohesive soils and pile groups which terminate above compressible deposits. Settlement magnitude may be computed using Figure 8-9 and the methods shown for spread footings. Time rate of settlement can be estimated using the formula

$$t = \frac{TH_v^{\ 2}}{C_v} \tag{6-4}$$

Where: $H_v$ is the maximum vertical drainage path in the clay layer(s) below the pile tips.
$C_V$ is the coefficient of consolidation.

## 8.9 NEGATIVE SKIN FRICTION

Engineers typically think in terms of a pile transferring load to a soil which may consolidate and cause settlement. However, another mechanism called negative skin friction (or drag) exists, which may be the cause of large foundation movement. In this case compressible soil surrounding the pile consolidates under external loads and moves downward relative to the pile. This relative movement causes negative skin friction to develop and the soil load to be applied to the pile rather than vice versa. The most common instance of negative skin friction development is where fill is placed over a compressible deposit after piles have been driven. Such situations, if unanticipated, can cause large settlement and/or soil bearing capacity failure of friction piles or structural failure of end bearing piles. Negative skin friction can be particularly severe when end bearing, battered piles are affected because of additional torsional forces applied to the piles.




Notes: 1. Plan area to outside of pile group = B × A.
2. For relatively rigid pile cap, pressure distribution is assumed to vary with depth as shown.
3. For flexible slab or group of small separate caps, compute pressures by elastic solutions
4. Compute settlements by methods of Chapter 6

Figure 8-9: Settlement of pile groups

Computation of negative skin friction for cohesive soil involves use of the method previously shown in 8.4.2. Granular soils which are in contact with piles above the compressible deposit, can cause large forces. These granular drag forces can be estimated using the method shown previously for pile skin friction in granular soils. The amount of relative settlement between soil and pile that is necessary to mobilize drag is about 1/2 inch. At that movement the maximum value of drag is equal to the soil adhesion or friction resistance. The drag cannot exceed these values because slip of the soil along the pile occurs at this value. It is particularly important in the design of friction piles to determine the depth below which the pile will be unaffected by negative skin friction. Only below that depth can positive skin friction forces contribute to safely supporting the pile loading.

In past years, additional piles were added to structures to carry suspected drag loads. Such practice was very expensive as each pile contributed only a small portion of its structural capacity toward support of the structure. Recently, methods have been developed to protect the pile from negative drag loads. The most cost-effective treatment is the application of a slip-layer of bitumen to the pile portion which will be embedded in the zone of drag. Bituminous coatings can reduce drag by up to 90 percent. The major problem is protecting the coating during pile installation; especially through coarse surface soils. An inexpensive method of protecting the bitumen is to weld an oversized collar around the pile where the bitumen ends. The collar opens an adequate size hole to permit passage of the bitumen for moderate pile lengths. Additional information is included in NCHRP Project Report 24-5, Downdrag on Bitumen Coated Piles. Example of specification of bitumen coating of piles are shown in appendix G and H.

## 8.10 DRILLED SHAFTS

A drilled shaft is a machine (and/or hand) excavated shaft in soil or rock that is filled with concrete and reinforcing steel, with the primary purpose of structural support. A drilled shaft is usually circular in cross section and may be belled at the base to provide greater bearing area. A typical drilled shaft is shown in Figure 8-11. Other terminology commonly used to describe a drilled shaft includes: drilled pier, drilled caisson and bored pile. Rectangular drilled shafts are called barrettes.

Vertical load is resisted by the drilled shaft in base bearing and side friction. Horizontal load is resisted by the shaft in horizontal bearing against the surrounding soil or rock.

Characteristics:

The following special features distinguish drilled shafts from other types of foundations:

1. The drilled shaft is installed in a drilled hole, unlike the driven pile.

2. Wet concrete is cast and cures directly against the soil forming the walls of the borehole. Temporary steel casing may be necessary for stabilization of the open hole and may or may not be extracted.

3. The installation method for drilled shafts is adapted to suit the sub-surface conditions.

Advantages and Disadvantages of Drilled Shafts

1. Advantages

    a. Construction equipment is normally mobile and construction can proceed rapidly.
    b. The excavated material and the drilled hole can often be examined to ascertain whether or not

the soil conditions at the site agree with the projected soil profile. For end-bearing situations, the soil beneath the tip of the drilled shaft can be probed for cavities or for weak soil if desirable.

c. Changes in geometry of the drilled shaft may be made during the progress of the job if the subsurface conditions so dictate. These changes include adjustment in diameter and in penetration and the addition or exclusion of underreams.

d. The heave and settlement at the ground surface will normally be very small.

e. The personnel, equipment, and materials for construction are usually readily available.

f. The noise level from the equipment is less than for some other methods of construction.

g. The drilled shaft is applicable to a wide variety of soil conditions. For example, it is possible to drill through a layer of cobbles and for many feet into sound rock. It is also possible to drill through frozen ground.

h. A single drilled shaft can sustain very large loads so that a cap may not be needed.

i. Data bases which contain documented load transfer information are available that allow confident designs of drilled shafts to be made considering load transfer both in end bearing and in side resistance.

j. Use in constricted areas. The shaft occupies less area than the footing and thus can be built closer to railroads and existing structures.

k. Drilled shafts may be more economical than spread footing construction, especially when the foundation layer is deeper than 10' below the ground or at water crossings.

2. Disadvantages

a. Construction procedures are critical to the quality of the drilled shaft. Knowledgeable inspection is required.

b. Drilled shafts are not normally used in deep deposits of soft clay or in situations where artesian pressures exist.

c. Static load tests to verify ultimate capacity of large diameter shafts are very costly.

d. Lack of general knowledge of construction and design methods has restricted the use of drilled shafts and in some instances has led to improper design.

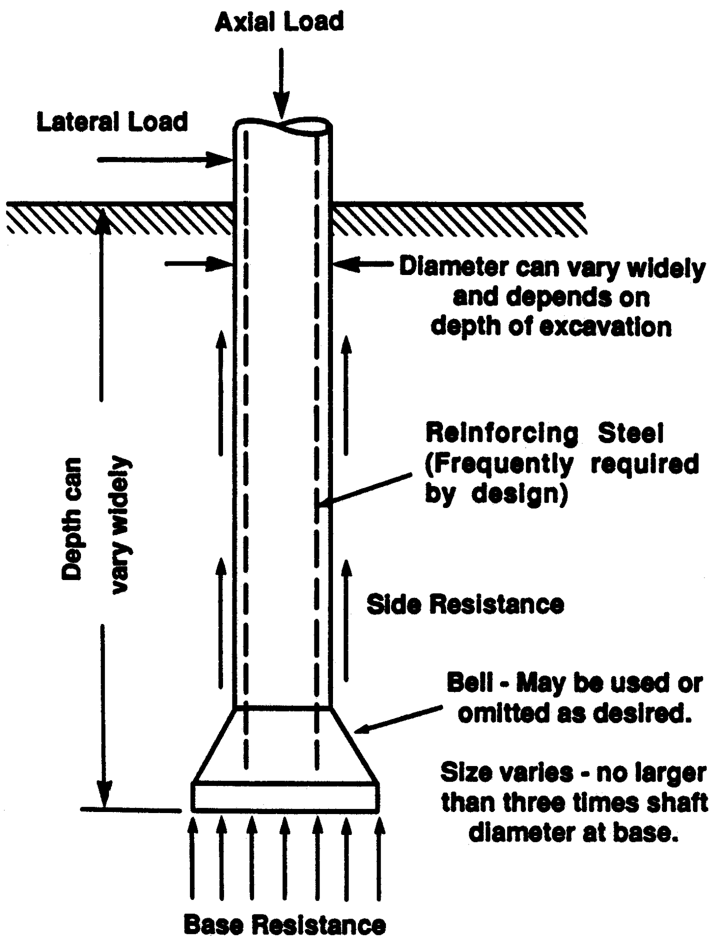



Figure 8-10: A typical drilled shaft

### 8.10.1 Design Procedure

Subsurface investigation for drilled shaft designs must include an assessment of the potential shaft construction methods as well as a determination of soil properties. The standard method for obtaining soil characteristics is similar to pile foundations and involves laboratory testing of undisturbed samples and the use of in situ techniques such as: the standard penetration test. Constructibility is difficult to assess from routine geotechnical investigations. Critical items such as hole caving, dewatering and obstructions can best be examined by drilling a full diameter test shaft hole during the exploration or design phase of the project. These test holes are usually done by local drilled shaft contractors under a short form contract from the highway agency. A detailed log should be made of the test hole including items such as type of drilling rig, rate of drilling, type of drill tools and augers used, etc. Such information should be made available for bidders. In addition, these test shaft holes may be cased with a "windowed" casing for inspection by designers and/or prospective bidders.

Subsurface Conditions Affecting Construction

a. The stability of the subsurface soils against caving or collapse when the excavation is made will determine whether a casing is necessary or not. The dry method of construction can be used only where the soils will not cave or collapse. The casing method must be used if there is danger of caving or collapse.

b. It must be determined if groundwater exists at the site and what rate of flow can be expected into a shaft excavation. The presence of groundwater will indicate if a tremie pour shaft will be needed or if a tremie seal must first be poured, the shaft dewatered, and then the remainder of the shaft poured in the dry. In either instance, the design must assure access to the top of the seal to allow the surface to be thoroughly cleaned prior to placing additional concrete. The shaft must be large enough to accommodate a worker or the top surface of a small diameter shaft seal must be located so that it is accessible.

c. Any artesian water conditions must be clearly identified in the contract documents. Artesian water flowing into a pour could spoil the concrete, or cause collapse or heaving of the soil at the excavation.

d. The presence of cobbles or boulders can cause difficulties in drilling. It is sometimes not easy to extract large pieces of rock, especially with the smaller diameter shafts.

e. The presence of existing foundations or structures.

f. Presence of landfill that could contain material that cannot be easily excavated, such as an old car body.

g. Presence of rock may require more sophisticated drilling methods or shooting with explosives.

h. Presence of a weak stratum just below the base of the drilled shaft. For this situation drilling may have to be extended below the weak stratum.

The total axial capacity of the drilled shaft is composed of two factors: the base capacity and the side capacity. The general formula is:

$$Q_T = Q_B + Q_S$$

Where: $Q_T$ = Total axial capacity of the foundation
$Q_B$ = Base capacity
$Q_S$ = Side capacity

The procedures for estimation of drilled shaft capacity have improved significantly in the past decade. The major reason for this change is a data base has been developed on load transfer in skin friction and in end bearing. It is now well established that drilled shafts can carry a substantial portion of applied loads in skin friction. As with pile foundations, the ultimate skin friction is mobilized at a small downward movement of the shaft relative to the soil. End bearing resistance is developed in relation to the amount of deflection at the tip.

Separate analyses are required to determine skin friction and end bearing contributions in different soil types and rock. Details of these analyses can be found in FHWA publication IF 99-025 "Drilled Shafts: Construction Procedures and Design Methods." The general step by step outline of the design procedure which has been excerpted from that publication follows.

### 8.10.2 Step-by-Step Procedure for Drilled Shaft Design

1. Clay and Sand

   a. Develop soil profile, and obtain location of water table from available data.

   b. Obtain undrained shear strength of clay from laboratory testing of undisturbed specimens and/or from in-situ tests. Undrained shear strengths should be either those obtained from unconsolidated, undrained (UU) triaxial compression tests or should be converted from other test results to values that would have been obtained, approximately, had UU triaxial compression tests been conducted. For example, limit pressure values ($P_L$) from the pressuremeter divided by a theoretical cavity-expansion factor (of about 6) will normally lead to excessively high values of $C_u$ and, ultimately, to unconservative design. Instead, ($P_L$) should be factored by a correlation factor that has been developed between $C_u$ from UU triaxial tests and $P_L$ for the soil formation under consideration.

   c. Obtain the N-values for sand from results of the Standard Penetration Test. These N-values are not to be corrected for fines or for overburden but should be the raw N-values obtained in the field.

   d. Review construction specifications and inspection procedures to ensure that high quality construction will be done.

   e. Obtain loadings for the drilled shafts, both axial and lateral. Take any possible downdrag into account.

   f. Select a factor of safety (or load and resistance factors), taking into account all of the pertinent information about the particular job. With good soil data the overall (global) factor of safety commonly ranges from 2 to 3.

   g. If clay exists at the ground surface:

      - Estimate the depth of the zone of seasonal moisture change and analyze for uplift.

      - If the depth of the zone of seasonal moisture change is more than 5 ft., consider eliminating skin friction to a depth greater than 5 feet.

      - If the lateral loads are significant, select a size and bending stiffness for the drilled shaft and compute the groundline deflection. If the computed deflection is more than 0.2 in., consider eliminating the clay skin friction to the first point of zero lateral deflection.

   h. Select the geometry of the drilled shaft and solve for the ultimate side and base resistances, employing appropriate equations for clays and sands.

      The ultimate base and side resistances are then divided by appropriate factors of safety and compared to the design load. For small-diameter shafts (base diameters less than 75 inches in clay or 50 inches in sand) or for shafts with base diameters that are large where reduced net ultimate base capacities ($q_{br}$) have been used, a global factor of safety can usually be applied. Otherwise, a partial factor of safety should be applied separately for side and end bearing values. Partial factors of safety should also be considered if there are significant differences in uncertainties of soil properties above the elevations of the bases of drilled shafts compared to those below the base elevation. Several design loading conditions must usually be considered, and the drilled shaft foundation should be sized for the most critical condition.

Where a global factor of safety is used, the design loads may be multiplied directly by the global factors of safety and compared with the sum of the ultimate side and base resistances to verify a trial geometry.

i. For the geometry selected and working load, compute short and long term settlements.

2. Rock

a. Perform subsurface explorations to obtain cores for laboratory strength testing, to obtain the RQD, to map the spacing and thickness of discontinuities, and to develop a profile of the subsurface conditions.

b. Obtain the compressive strength of the rock cores and the Young's modulus. If feasible, use the pressuremeter to obtain the Young's modulus of the rock mass.

c. Set up construction specifications to ensure a proper excavation, that excess loose material is removed when end bearing is part of design, and that the sides of the socket are roughened.

d. Obtain design loadings for the drilled shaft, both axial and lateral.

e. Select a global factor of safety, taking into account the fact that detailed information on discontinuities is very difficult to obtain and that the behavior of a drilled shaft is strongly influenced by the nature of the discontinuities.

f. Obtain values of ultimate loads by multiplying the design loads by the selected global factor of safety.

g. Select the trial geometry of the drilled shaft.

h. If the rock is weak (compressive strength of less than 750 psi), the design should depend on load transfer in side resistance. The design should follow procedures in FHWA IF 99-025 for load transfer in intermediate geomaterials. The settlement should be checked to see that it does not exceed 0.4 inches.

i. If the rock is strong, the design should be made on the basis of end bearing. The settlement under working load should not exceed the allowable settlement as dictated by the superstructure.

j. A load test program should be considered if there are serious questions about the quality of the rock.

### 8.10.3 Construction Methods

1. Dry Method

The dry method is applicable to soils above the water table that will not cave or slump when the hole is drilled to its full depth. A soil that meets this specification is a homogeneous stiff clay. The dry method can be employed in some instances with sands above the water table if the sands have some cohesion, or if they will stand for a period of time because of apparent cohesion.

The dry method can be used for soils below the water table if the soils are low in permeability so that only a small amount of water will seep into the hole during the time the excavation is open.

The dry method consists of drilling a hole using an auger or bucket drill, without casing, placing a rebar cage and then filling the hole with concrete.

2. Casing Method

The casing method is applicable to sites where soil conditions are such that caving or excessive deformation will occur when a hole is excavated. An example of such a site is a clean sand below the water table.

This method employs a cylindrical (usually steel) casing inside the hole to hold back the caving soil. The excavation is made by driving, vibrating, or pushing down a heavy casing to the proposed founding level and by removing the soil from the casing either continuously as driving proceeds or in one sequence after the casing has reached the founding level. The casing is sometimes left in the hole after the concrete is poured.

3. Slurry Displacement Method

This method is gaining in popularity. A bentonite slurry is introduced into the excavated hole to prevent caving or deformation of loose or permeable soils. Drilling continues through the slurry using an auger or clamshell mounted on a Kelly bar. When the desired depth is reached the rebar cage is lowered into the slurried hole. Concrete is then tremie-poured into the hole. Slurry is displaced by the heavier concrete and collected at the surface in a sump. The slurry may again be used in another hole.

### 8.10.4 Drilled Shaft Publications

FHWA publication "Drilled Shafts: Construction Procedures and Design Methods," FHWA-IF 99-025 contains information on vertical loading. FHWA publication, “Load Transfer for Drilled Shafts in Intermediate Geomaterials”, FHWA RD 95-172 contains information for vertical on weak rock. For lateral load information, consult either the "Handbook on Piles and Drilled Shafts Under Lateral Load"' FHWA-IP-84-11 or COM624P User's Manual Version 2.0, FHWA-TA-91-048. Consult Drilled Shafts: Construction Procedures and Design Methods, FHWA HI-88-042, for additional information on construction.

## 8.11 APPLE FREEWAY DESIGN EXAMPLE – PILE DESIGN

In this chapter the Apple Freeway is used to illustrate the pile design for support of the pier and abutment. Although drilled shafts may also be a feasible deep foundation design alternate for this structure, the details of the drilled shaft design are beyond the scope of this manual. The computation process for static analysis to determine pile capacity by Nordlund Method is presented along with the computation of pile driving resistance.

| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | Po Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surcharge |
| → | **Pile Design** |
| Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment Instrumentation |

Design Soil Profile
Static Analysis – Pier
- Pipe Pile
- H – Pile

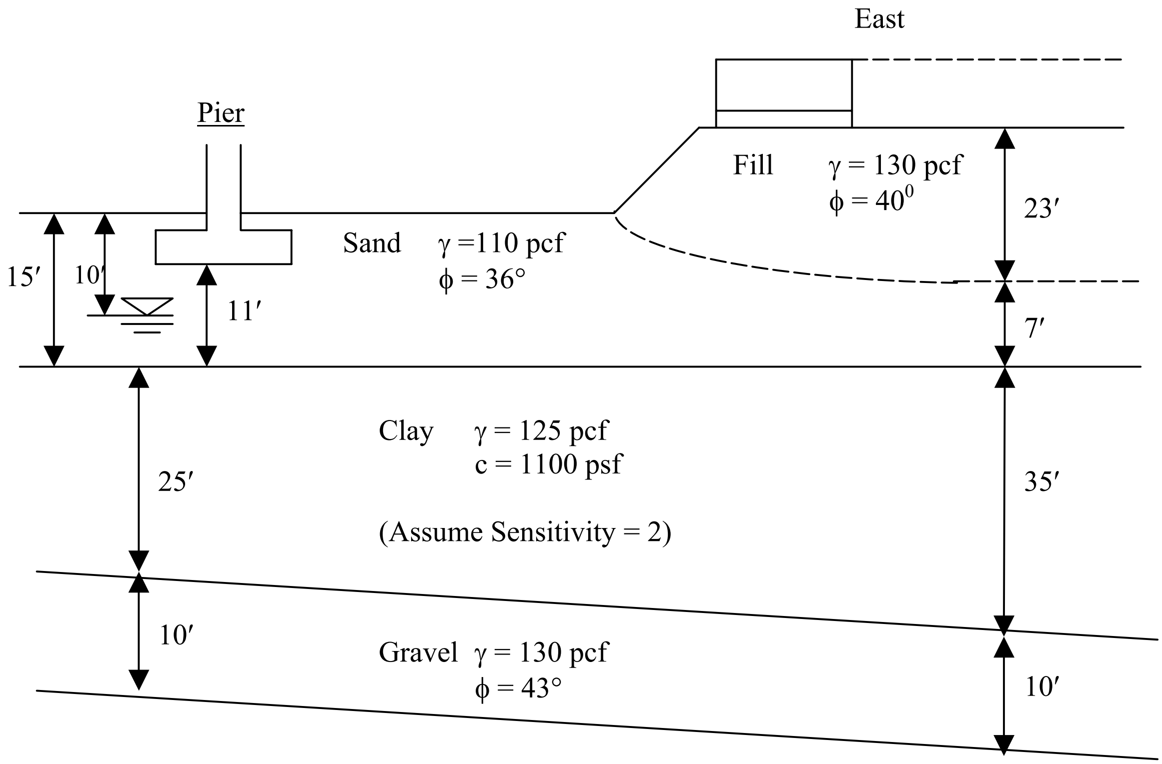
Static Analysis – abutment
- Pipe Pile
- H – Pile

Driving Resistance
Abutment Lateral Movement

Apple Freeway Design Example – Pile Design
Exhibit A

**Given:** The subsurface profile and soil properties shown below.

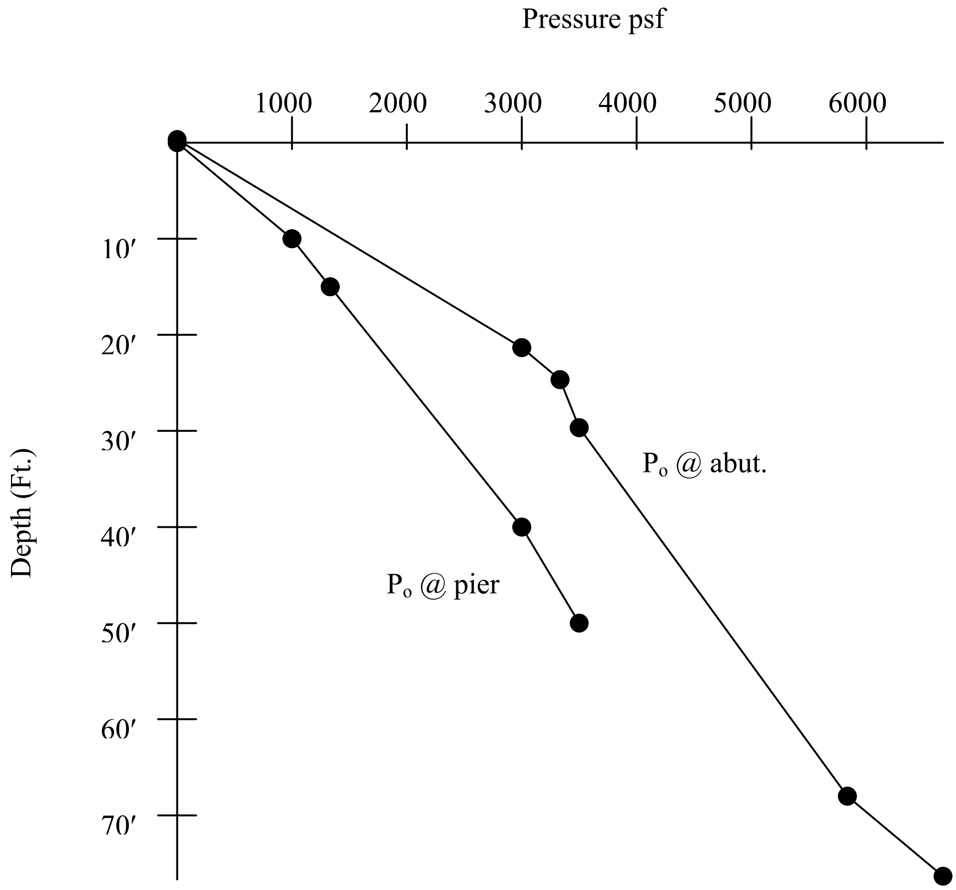
**Required:** Determine the allowable pile capacity using static analysis.




**Consideration for Pile Type Selection:**

- Spread footings would be feasible at both the pier and abutment except for settlements due to consolidation of the clay deposit. To eliminate these settlements any pile type selected must achieve capacity below the bottom of the clay deposit.
- End bearing will provide most of the ultimate resistance at either the pier or abutment location due to the minimal thickness of the gravel layer.
- The maximum estimated structural load of 2200 tons can be supported by either the gravel or the rock layer. However pile driveability appears to be an issue in design.
- Required loads, end bearing support, and difficult driving concerns would favor a straight-sided steel pile over either a timber or concrete or tapered pile.
- Static analyses will be used to determine if a displacement pile with an end plate or a non-displacement pile will provide the best choice for both bearing and driveability. The designer selected a 12” diameter closed end pipe pile and a 12” H-pile for alternate evaluation at this site. The structural engineer usually designs the pipe pile for a 70-ton design load and the H-pile for a 120-ton design load.

**Step 1:** **Plot $P_o$ diagram.**




**Static Pile Analysis – Pier**

**<u>A. For 12″ diameter pipe pile (Closed end, 70 ton design)</u>**

**Step 2A:** **Compute skin resistance.**

- Sand Layer

4′ - 15′ Use Nordlund Method

D = 15 – 4 = 11′

$$V = \frac{\text{pile vol.}}{\text{foot}}$$

$$= \frac{\pi d^2}{4} = 0.785 \,{}^{CF}\!/_{Ft.}$$

$$\delta/\phi = 0.6$$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

Corr. Factor = 0.75

$C_d = \pi\ d = \pi\ (1) = 3.14'$

$P_0$ Avg. @ $\dfrac{4+15}{2} \sim 9.5' = 1050$ psf

$q_s = K_\delta\ (C_F)(P_0)(C_d)(\text{Sin}\ \delta)D$
$\quad = (1.75)(0.75)(1050)(3.14)(\text{Sin}\ 21.6)\ 11$
$q_s = 8.7$ tons

- Clay Layer

15′ - 40′

$q_s = C_aC_dD$

$C_a$ (Adhesion) ≅ 1100 psf

$q_s = 1100\ (3.14)\ 25 = 43.1$ tons

- *Gravel Layer (Try 4′ Embedment)

*Remember to reduce $\phi$ of 43° to maximum 36° value for hard, angular gravel skin friction.
40′ - 44′.

$q_s = (K_\delta)(C_F)(P_o)(C_d)(\text{Sin}\ \delta)D$

$\delta/\phi = 0.6$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

$C_F = 0.7$

$P_0 = 3200$ psf

$q_s = (1.75)(0.7)(3200)(3.14)(\text{Sin}\ 21.6°)4$
$q_s = 9$ tons

**Step 3A: Compute end bearing.**

- Gravel Layer

$P_o$ Max.

a. $Q_p = A_p \alpha P_D N'_q$
$= (0.785)(0.75)(3000)(300)$
$Q_p = 265$ tons

b. $Q_{lim}$ = (Limiting Point Resist.) $\times A_p$
$= (320 \text{ tsf})(0.785)$
$Q_{lim} = 251$ tons

$\therefore Q_p = 251$ tons

It is obvious that any embedment in gravel layer will produce capacities > 200 tons. Therefore, estimate pile length to top of gravel.

**Step 4A: Determine driving resistance (For 70 ton load with SF = 2).**

$$Q_{drive} = q_{sand} + \frac{q_{clay}}{\text{sensitivity}} + (70^T \times 2)$$

$$= 8.7^T + \frac{43.1^T}{2} + 140^T$$

$Q_{drive} = 170^T$

**B. For *12″ H-Pile (120 ton Design Load)**
***Assume 12x84 H-Section**

**Step 2B: Compute Skin Resistance**

- Sand Layer

4′ - 15′

$$V = \frac{24.6}{144} = 0.17 \, {}^{CF}\!/_{Ft.}$$

$$\delta/\phi = 0.80$$

$\delta = (0.80)(36°) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$C_d = 4'$

$P_0 = 1050$

$q_s = (1.30)(0.92)(1050)(4)(\text{Sin } 28.8^{o})\ 11$
$q_s = 13.0$ tons

- Clay Layer

15′ - 40′

$q_s = C_a C_d D$

$C_a = 1100$ psf
$q_s = 1100\ (4)\ 25 = 55$ tons

- *Gravel Layer (Try 4′ Embedment)

*Use $\phi_{Max} = 36°$

40′ - 44′

$q_s = (K_\delta)(C_F)(P_0)(C_d)(\text{Sin } \delta)D$

$$V = \frac{24.6}{144} = 0.17\ {}^{CF}\!/_{Ft.}$$

$${}^{\delta}\!/_{\phi} = 0.80$$

$\delta = (0.80)(36°) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$C_d = 4′$

$P_0 = 3200$ psf

$q_s = (1.30)(0.92)(3200)(4)(\text{Sin } 28.8°)4$
$q_s = 14.7$ tons

**Step 3B: Compute End Bearing** (Use $\phi = 43°$) at 44′

a. $Q_p = A_p \alpha P_D N'_q$

$$= \frac{24.6}{144}(0.78)(3000)(300)$$

$Q_p = 60$ tons

b. $Q_{lim} = q_{lim}A_P$

$$= (320 \text{ tsf}) \frac{24.6}{144}$$

$Q_{lim}$ = 54.7 tons

$\therefore$ $Q_p$ = 54.7 tons

- Total useable soil capacity below clay is = $14.7^T + 54.7^T$ = 69.4 Tons
- Total Required capacity is 240 Tons
- Extending pile to 50′ only increases $Q_s$ to $37^{Tons}$

Conclusion: Pile must bear on rock to develop 240 Tons capacity below clay layer. Therefore estimate pile length to rock.

**Step 4B: Compute H – Pile Driving Resistance**

$$Q_{driving} = Q_{lim} + \frac{q_{clay}}{sensitivity} + (120T \times 2)$$

$$= 13 + \frac{55}{2} + 240$$

$Q_{driving}$ = 280.5 Tons

* Composed of 37 t skin friction in the gravel and 203 t in end bearing on rock.

**STATIC PILE ANALYSIS - ABUTMENT @ STA 93 + 50**

**<u>A. For 12″ Diameter Pipe Pile</u>**

**Step 2A & 3A:** Based on computation at the pier, the pipe pile will develop the 140 ton ultimate load at the top of the gravel layer, ie. an estimated length of 65′. However the driving resistance will increase.

**Step 4A: Compute driving resistance.**

- Fill (Use $\phi_{Max}$ = 36°)

$q_s = (K_\delta)(C_F)(P_0)(C_d)(\text{Sin } \delta)\ C_d\ D$

V = 0.785 CF/Ft.

$\delta/\phi = 0.6$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

$C_F = 0.75$

$P_0 = 1650$ psf

$q_s = (1.75)(0.75)(1650)(3.14)(\text{Sin } 21.6°)23$
$q_s = 28.8$ tons

- Sand

$q_s = (1.75)(0.75)(3330)(3.14)(\text{Sin } 21.6°)7$
$q_s = 17.5$ tons

- Clay

$$q_s = \frac{C_a C_d d}{\text{sensitivity}}$$

$$q_s = \frac{(1100)(3.14)(1)35}{2} = 30.2 \text{ tons}$$

Driving resistance @ top of gravel 28.8 + 17.5 +30.2 + 140 = 216.5 tons

**Step 5A: Check driving resistance in embankment.**

Assume pile tip embedded 23′

$q_s = 28.8$ tons (from Step 4A)

$Q_p \quad = A_p \alpha P_{0\,23'} N'q$
$\quad\quad = (0.785)(0.74)(2990)170$
$Q_p \quad = 147.6^{\text{tons}} < q_{lim} = 200(0.785) = 157$ tons

$Q_{Drive\ Emb} = 28.8 + 147.6 = 176.4$ tons

* Pre augering may be required.

**<u>B. 12″ H – Pile (120$^T$ design) -- 12×84 section</u>**

Steps 2B & 2C Estimate length to rock ie. 75′, as pier computation showed H-pile must bear on rock to achieve designed ultimate capacity.

**Step 4B: Compute driving resistance.**

- Fill

V = 0.17 CF/Ft.

$\delta/\phi = 0.80$

$\delta = (0.80)(\underset{\phi_{max}}{36°}) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$q_s = (1.30)(0.92)(1495)(4)(\text{Sin } 28.8°)23$
$q_s = 39.6$ tons

- Sand

$q_s = (1.30)(0.92)(3303)(4)(\text{Sin } 28.8°)7$
$q_s = 26.6$ tons

- Clay

$$q_s = \frac{C_a C_d d}{\text{sensitivity}}$$

$$q_s = \frac{(1100)(4)35}{2} = 38.5 \text{ tons}$$

- Gravel

$q_s = (1.30)(0.92)(5600)(4)(\text{Sin } 28.8°)10$
$q_s = 64.5$ tons

$Q_{Drive} = 39.6 + 26.6 + 38.5 + [\underbrace{64.5 + 175.5}_{240^T}]$

$Q_{Drive} = 344.7$ tons

**Step 5B: Check H – Pile driving resistance in embankment**

Assume pile tip embedded 23′

$q_s = 39.6$ tons (From Step 4B)

$q_p = A_p \alpha P_{0\,23'} N'q$

$$q_p = \frac{24.6}{144}(0.74)(2990)170$$

$q_p = 32.1 \text{ tons} < q_{lim} = (200)(0.17) = 34$ tons

$Q_{Drive\ Emb.} = 39.6^T + 32.1^T = 71.7$ tons

* Preaugering may not be required.

**<u>Summary of the Pile Design Phase for the Apple Freeway Design Problem</u>**

- <u>Design Soil Profile</u>

  Strength value selected for all layers.

- <u>Static Analysis - Pier</u>

  12" - 70 T Pipe Pile - 36' length required
  12" - 120 T H-Pile - 46' length required.

- <u>Static Analysis Abutment</u>

  12" - 70 T Pipe Pile - 65' length required
  12" - 120 T H-Pile - 75' length required.

- <u>Driving Resistance</u>

  Driving Resistances computed for both pipe and H-piles to permit design check of pile section overstress.

  Pipe pile will require pre-augering through embankment.

- <u>Abutment Lateral Movement</u>

  3" possible horizontal movement even with a pile foundation unless recommended waiting period observed prior to pile driving.

# CHAPTER 9.0
# CONSTRUCTION MONITORING AND QUALITY ASSURANCE

The successful transfer of design objectives into construction is accomplished by consideration of construction operations during the design phase. In recent years the amount of coordination between design and construction has steadily decreased; primarily due to graduate engineers who specialize in design and who are never exposed to construction operations. In past years, engineers either began their careers in construction and advanced into design, or were assigned the design and construction responsibilities for projects. Present lack of coordination stemming from inexperience with field operations can result in a technically superior set of construction plans and specifications, which cannot be built. Rational construction control is vital to assure a safe, cost-effective foundation and to avoid unnecessary court of claims actions.

## 9.1 EARTHWORK AND SPREAD FOOTING FOUNDATIONS

Approach embankment construction should be clearly defined in standard drawings as to materials and limits of placement. Such standards assure uniformity in construction due to the familiarity of the construction personnel with the operations being performed and results expected. The designer should keep to the standard unless major changes are required. Attempts at small changes in materials or limits are counterproductive to good construction.

The philosophy of approach embankment compaction is to insure adequate bearing capacity for abutments (or piers) placed in the embankment and to minimize settlement of the pavement or footing. Typical highway embankments require compaction to 90 percent of maximum density (AASHTO T180) to control pavement settlement. Designers of approach embankments should specify 95 percent of T180 to limit differential settlement between the structure and fill. If piles are used to support footings in fill, the maximum top size of embankment material should be limited to 6 inches to ease pile installation. If spread footings are used, a minimum of 5 feet of select material compacted to 100 percent of T99 should be placed beneath the footing and extended to beyond the wingwalls. This layer provides uniform support for the footing and a rigid transition between the structure-fill interface to minimize differential settlement. Construction control is usually keyed to percent compaction on the standard design drawings.

Construction of spread footings on soil must be controlled such that a stable surface exists on which to pour the footing. The designer should anticipate situations where construction operations may temporarily disturb the foundation soil, i.e., footing elevation in fine sand near the water table. A note should be included to alert the engineer of the potential problem and what action to take, i.e., if unstable soil is encountered at footing level, undercut one foot and backfill with gravel to footing level.

Assurance of the footing being placed on the proper soil can be guaranteed by including a soil profile in the contract plans and requiring inspection of the prepared footing level by either a representative of the geotechnical engineer or a construction inspector who can confidently verify actual foundation conditions.

Spread footing excavations frequently require sheeting to retain the excavation walls while the footing is poured. In cases where sheeting extends below footing level within three feet of the footing sides, consideration should be given to leaving the sheeting in place if:

1. Uniform footing settlement magnitude is critical, i.e., less than 1/2 inch.

2. Sheet pile Z - sections are used (because a large quantity of soil may remain stuck between adjacent flanges and be removed with sheeting).

3. Sheeting will only be pulled on one side (can cause differential settlement).

## 9.2 EMBANKMENT CONSTRUCTION MONITORING - INSTRUMENTATION

The observational approach to design involves monitoring subsoil behavior during early construction stages to predict responses to subsequent construction. Basic soil mechanics concepts can be used to accurately predict future subsoil behavior if data from instrumentation is analyzed after initial construction loads have been placed. Occasionally a design problem arises which is unique or of major criticality that can only be safely solved by utilizing the observational approach.

Embankment placement must be carefully observed and monitored on projects where stability and/or settlement are critical. The monitoring should include visual observation by the construction inspection staff and use of instrumentation. Without the aid of various forms of instrumentation, it is impossible to determine what is happening to the foundation. Instrumentation can be used to warn of imminent failure or to indicate whether settlement is occurring as predicted. The type of instruments to be used and where they will be placed should be planned by a qualified soils engineer. Actual interpretation and analysis of the data from the instrumentation should also be done by someone with a background in soil mechanics; however, the project engineer and inspector should understand the purpose of each type of instrumentation and what the data is to be used for.

### 9.2.1 Inspector's Visual Observation

In areas of marginal embankment stability, the inspector should walk the surface of the embankment daily looking for any sign of cracking or movement. Hairline cracks often develop at the embankment surface just prior to failure. If the inspector should discover any such indication, all fill operations should cease immediately. All instrumentation should immediately be read. The soils engineer should be notified. Subsequent readings will indicate when it is safe to resume operations. Unloading by removal of fill material is sometimes necessary to prevent an embankment failure.

### 9.2.2 Types of Instrumentation

The usual instrumentation specified to monitor foundation performance on projects where stability and settlement are critical consists of:

1. Slope Inclinometers are used to monitor embankment stability. A slope inclinometer consists of a 2 to 3-inch grooved plastic or metal tube that is installed in a borehole. The bottom of the slope inclinometer tube must be founded in firm soil or rock. A readout probe is lowered down the tube and deflection of the tube is measured. With a slope inclinometer, the amount and location of horizontal movement in the foundation soil can be measured. For embankments built over very soft subsoils, telescoping inclinometer casing should be used to account for vertical consolidation. In soft ground conditions, several inches of lateral movement (squeeze) may occur without shear failure as the embankment is built. Therefore, from a practical construction control standpoint, the rate of movement rather than the amount is the better indicator of imminent failure. Slope

inclinometer readings should be made often during the critical embankment placement period (daily if fill placement is proceeding rapidly) and readings should be plotted immediately on a time versus movement plot. Fill operations should cease if a sudden increase in rate of movement occurs.

2. Piezometers indicate the amount of pressure build-up within the water-saturated pores of the soil. There are critical levels to which the water pressure in the subsoil will increase just prior to failure. The soils engineer can estimate the critical water pressure level during design. Normally, the primary function of piezometers during fill placement is to warn of failures. Once the embankment placement is complete, the piezometers are used to measure the rate of consolidation. There are several different types of piezometers. The simplest is the open-standpipe type, which is essentially a well point with a metal or plastic pipe attached to it. The pipe is extended up through the fill in sections as the fill height increases. This type has the disadvantage that the pipes are susceptible to damage if hit by fill construction equipment. There are several types of remote piezometers that eliminate the requirement for extending a pipe up through the fill. Also the response time of open well piezometers is often too slow in soft clays to warn of potential embankment failure. The remote units consist of a piezometer transducer that is sealed in a borehole with leads carried out laterally under the base of the embankment to a readout device, which measures the porewater pressure. Pneumatic or vibrating wire piezometers have rapid response to changes in pore pressure.

3. Settlement devices are used to measure the amount and rate of settlement of the foundation soil due to the weight of the embankment. They are installed on or just below the existing ground surface before any fill is placed. The simplest settlement device is a settlement platform (usually a 3 or 4 foot square plywood mat or steel plate) with a vertical reference rod (usually 3/4-inch pipe) attached to the platform. The reference rods are normally added 4 feet at a time as the height of the embankment increases. The elevation of the top of the reference rod is surveyed periodically to measure the foundation settlement. Remote pneumatic settlement devices are also available. As with the remote piezometer devices, the remote settlement devices have the advantage of not having to bring a reference rod up through the fill.

### 9.2.3 Typical Instrument Locations

Instrument installations should be spaced approximately 250-500 feet along the roadway alignment in critical areas. Typical instrument locations for an embankment over soft ground are shown in Figure 9-1:

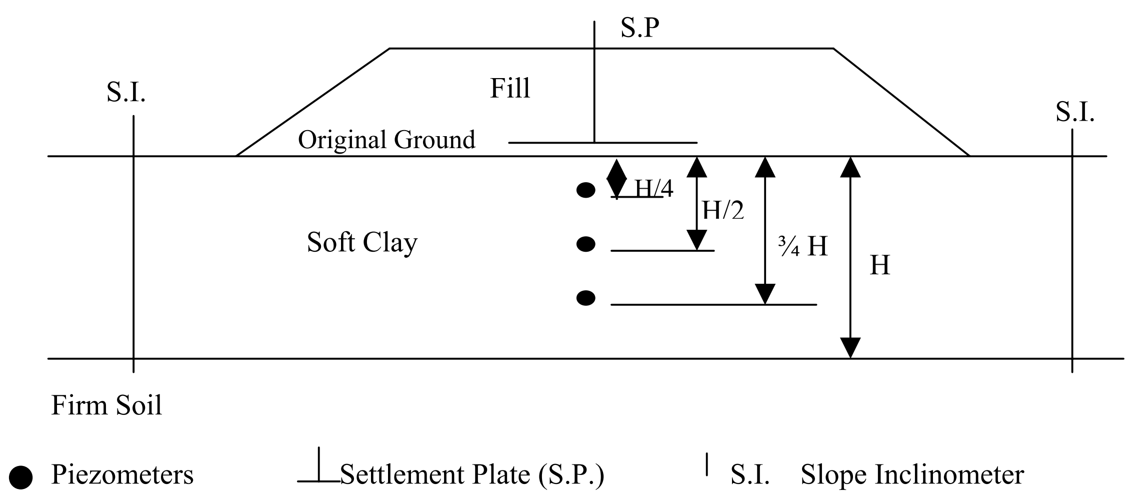



Figure 9-1: Typical instrumentation location for an embankment over soft ground

## 9.3 PILE FOUNDATIONS

Construction control of pile operations is a much more difficult proposition than for spread footings. During footing placement an inspector can easily examine a prepared footing area and watch the concrete footing being poured to assure a quality foundation. Piles derive their support below ground. Direct quality control of the finished product is not possible. Therefore, substantial control must be maintained over the peripheral operations leading to the incorporation of the pile into the foundation. In general terms, control is exercised in three areas; the pile material, the installation equipment, and the subsurface soil resistance. These items are interrelated with changes in one affecting the others. It is mandatory that pile foundation installation be considered during design to insure that the piles shown on the plans can be installed.

### 9.3.1 SELECTION OF DESIGN SAFETY FACTOR BASED ON CONSTRUCTION CONTROL

The safety factor to be selected for a pile foundation depends on both design aspects and construction control techniques. End bearing piles may be designed with a safety factor of 2.0 if adequate control is exercised in pile section design and hammer approval. Assuming that an adequate foundation investigation and static analysis has been performed for a friction pile foundation, the following safety factors are generally recommended to be applied to ultimate capacities of piles supported by soil based on construction control techniques to be used.

| Construction Control Technique | Global Safety Factor |
|---|---|
| Representative static load test | 2.00 |
| Representative dynamic load test | 2.25 |
| Wave equation analysis and indicator piles | 2.50 |
| Wave equation | 2.75 |
| Dynamic formula | 3.50 |

Piles supported primarily by end bearing on rock with RQD > 50 may use a safety factor of 2.0 assuming driveability has been confirmed by wave equation analysis. End bearing piles on poorer rock or intermediate geomaterials should use safety factors which consider both construction control and the previous history of end bearing piles in that particular formation.

### 9.3.2 RESPONSIBILTY FOR QUALITY ASSURANCE

Clear lines of responsibility are needed to permit successful installation of pile foundations. The designer is generally responsible for selection and display on the plans of the following information:

1. Pile details

    a. Material - concrete, steel or timber

    b. Allowable stresses - design and driving

    c. Cross section - diameter, tapered or straight, and wall thickness

d. Estimated length

e. Pile design load

2. Soils data

a. Subsurface profile

b. Soil resistance to be overcome to reach estimated length (as determined by static pile analysis).

c. Special notes - boulders, artesian pressure, buried obstructions, etc.

3. Pile Installation

a. Method of hammer approval

b. Special notes - spudding, preaugering, jetting, reinforced tips, etc.

c. Estimated blow count at estimated length

The engineer on construction is generally responsible for quality control of the following items and for initiating communication with the designer on variations from the plans.

1. Pile

a. Quality control testing or certification of materials.

2. Soils data

a. Major discrepancies in soil profile reported to designer as soon as encountered.

3. Installation

a. Hammer maintained in good working order - cushion replaced regularly.

b. Final pile length determined from estimated blow count, estimated length and subsurface profile.

c. Pile stress controlled.

d. Documentation of field operations.

Proper construction control of pile driving requires good communication between designer and field engineer. Such communication cannot follow traditional lines and still be effective. Answers are needed in a short time to prevent expensive contractor down time or to prevent pile driving from continuing in an unacceptable fashion.

Good communication should begin with a preconstruction meeting of the foundation designer and field engineer. The designer should briefly explain the design and point out possible problem areas. The most important purpose is to establish a direct line of communication between the designer and the field. The designer should advise the field engineer, on request, of the design aspects of problems occurring in construction. The ultimate decision on any field problem must remain in the traditional lines of authority established for construction. Interaction between office and field will simplify and expedite decisions.

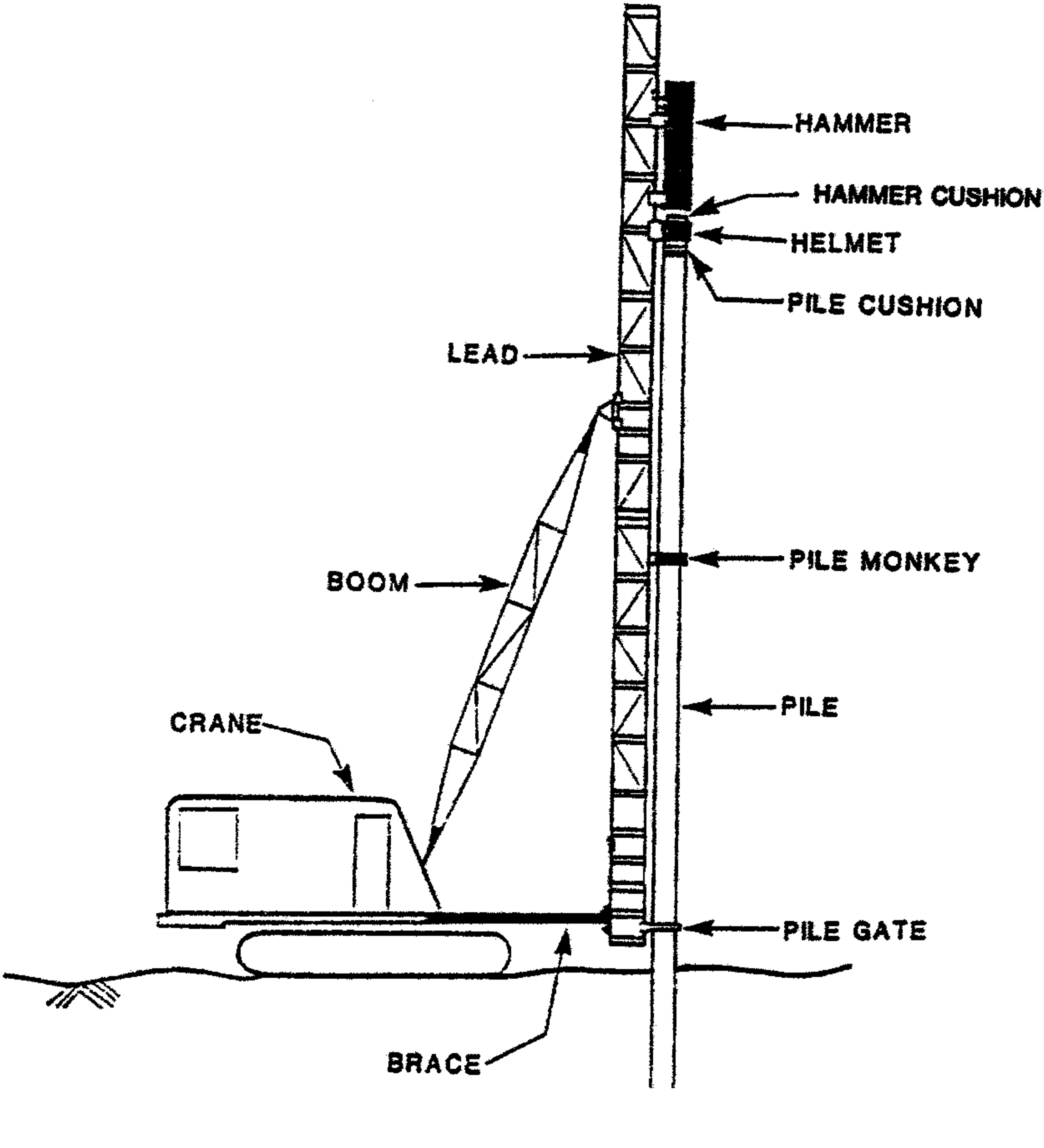
## 9.4 PILE DRIVING EQUIPMENT AND OPERATION

Proper inspection of pile driving operations requires that the inspector have a basic understanding of pile driving equipment. Estimation of "as driven pile capacity" is usually based on the number of hammer blows needed to advance the pile a given distance. Each hammer blow transmits a given amount of energy to the pile. The total number of blows is the total energy required to move the pile a given distance. This energy can then be related to soil resistance and supporting capacity. However, pile inspection entails more than counting blows of the hammer.

The energy transmitted to the pile by a given hammer can vary greatly depending on the equipment used by the contractor. Energy losses can occur by poor alignment of the driving system, improper or excessive cushion material, improper appurtenances or a host of other reasons. As the energy losses increase, more blows are required to move the pile. The manufacturer's rated hammer energy is based on minimal energy losses. Assumptions that the hammer is delivering its rated energy to the pile can prove dangerous if substantial energy is lost in the driving system. Artificially high blow counts can result in acceptance of driven pile lengths, which are shorter than that necessary for the required pile capacity.

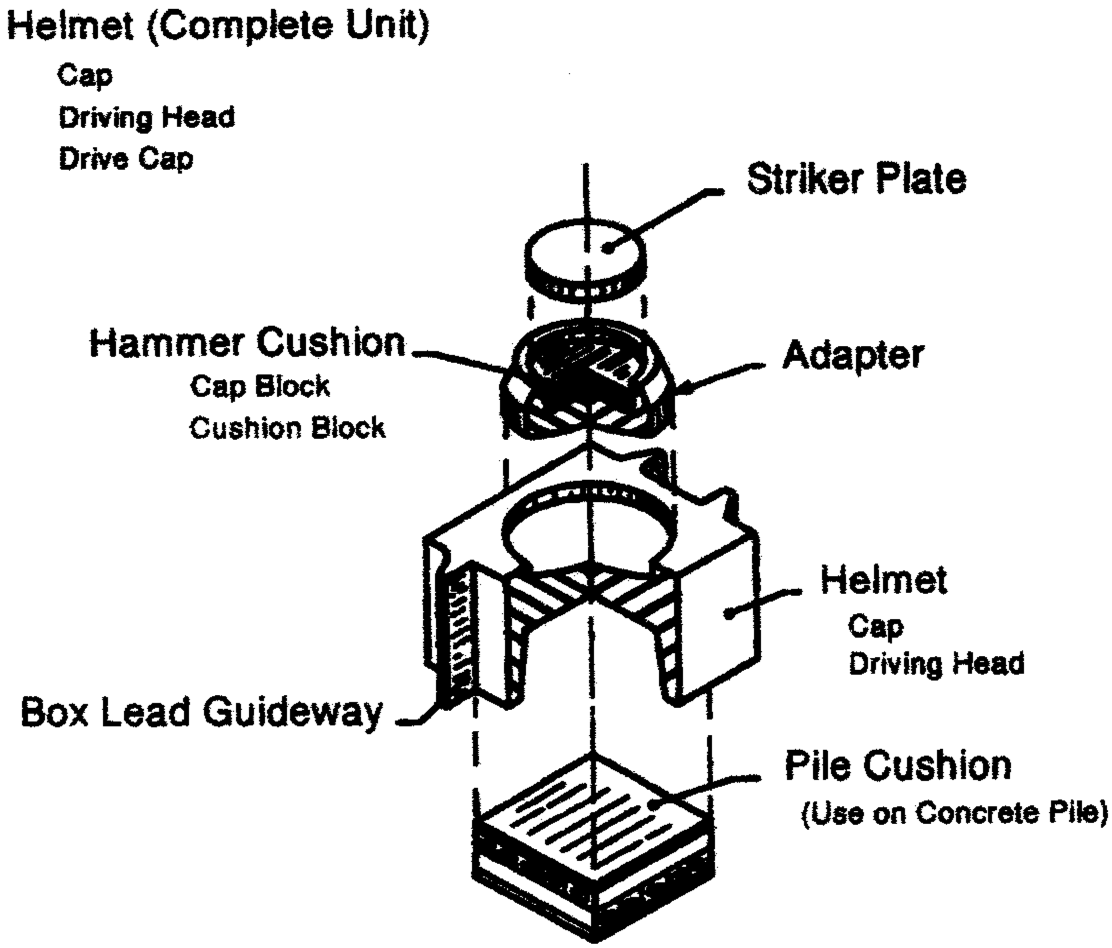
Important elements in the driving system include the leads, the hammer cushion, the helmet, and for concrete piles, the pile cushion. Typical components of a pile driving system are shown in Figure 9-2. The leads are used to align the hammer and the pile such that every hammer blow is delivered concentrically to the pile system. The helmet holds the top of the pile in proper alignment and prevents rotation of the pile during driving. Typical components of a helmet are shown in Figure 9-3. Both the hammer and the helmet ride in the leads so that hammer - pile alignment is assured. Inspectors should be concerned about "flying leads" which sit on top of the pile and only control the hammer alignment. Erratic energy delivery to the pile can be caused by misalignment of such lead systems

.




**Pile Support System**

Figure 9-2: Typical components of a pile driving system




**Note: The helmet shown is for nomenclature only. Various sizes and types are available to drive H, pipe, concrete (shown) and timber piles. A system of inserts or adapters is utilized up inside of the helmet to change from size to size and shape to shape.**

Figure 9-3: Typical components of a helmet

All impact pile driving equipment, except some gravity hammers shall be equipped with a suitable thickness of hammer cushion material. The function of this material is to prevent damage to the hammer or pile and insure uniform energy delivery per blow to the pile. Hammer cushions shall be made of durable manufactured materials provided in accordance with the hammer manufacturer's guidelines, except that all wood, wire rope and asbestos hammer cushions are specifically disallowed and shall not be used. The thicker the hammer cushion, the less the energy that is transferred to the pile. Mandatory use of a durable hammer cushion material, which will retain uniform properties during driving, is necessary to accurately relate blow count to pile capacity. Non-durable materials, which deteriorate during driving cause erratic energy delivery to the pile and prevent the use of blow counts to determine pile capacity.

The heads of concrete piles shall be protected by a pile cushion made of plywood. The minimum thickness of pile cushion placed on the pile head shall not be less than four inches. A new pile cushion should be provided for each pile.

A non-routine element called a follower may be used in the driving system, particularly for piles driven below water. Followers cause substantial and erratic reduction in the hammer energy transmitted to the pile due to the follower flexibility, poor connection to the pile head, frequent misalignment, etc. Reliable correlation of blow count with pile capacity is impossible when followers are used. Special monitoring with devices such as the pile analyzer is required when followers are used.

## 9.5 DYNAMIC PILE DRIVING FORMULAE

In the 1800's the fundamental pile driving formula was established to relate dynamic driving forces to available pile bearing capacity. The formula was based on a simple energy balance between the kinetic energy of the ram at impact and the resulting work done on the soil, i.e., a distance of pile penetration against a soil resistance. The concept assumed a pure Newtonian impact with no energy loss. The fundamental formula was expressed as follows:

KINETIC ENERGY INPUT = WORK DONE ON SOIL

$$\therefore WH = RS \qquad (9\text{-}1)$$

Where:
- W = Weight of the ram
- H = Distance of ram fall
- R = Total soil resistance (driving capacity) against the pile
- S = Pile penetration (set) per blow

Using this simple energy approach, the total soil resistance (driving capacity) could be calculated as:

$$R = \frac{WH}{S} \qquad (9\text{-}2)$$

An inherent difficulty in the pile driving operation is that a small portion of the ram's kinetic energy actually causes penetration of the pile. Studies indicate that typically only 30 to 65 percent of the rated energy is passed thru to the pile. Much energy is lost in either heat (soil friction, hammer mechanism, pile material, etc.) or strain (elastic compression of the cushion, the pile and the surrounding soil. An attempt was made to quantify these losses in the late 1800's by observing pile driving operations. The result was the ENR pile driving formula which related the safe load that a pile could withstand to the input energy and set per blow. Basically the equation was developed to lump all losses and safety factors into a single factor. The formula follows:

$$P = \frac{2WH}{S + k} \qquad (9\text{-}3)$$

Where:
- P = Safe pile load in kips
- W = Weight of ram in kips
- H = Distance of ram fall in feet
- S = Set per blow in inches
- k = Constant which varies from 0.1 to 1 based on hammer type

Observe that the ENR formula is not dimensionally correct as H is in feet and S is in inches. The overall safety factor can be roughly determined by comparing the ENR formula for safe load to the fundamental formula (which is for total driving capacity) as follows:

1. Change ENR to dimensionally correct form by changing H from feet to inches,

$$P = \frac{2WH' \times 12}{(S'' + k) \times 12} \qquad \text{i.e., H (feet) x 12 = H'' (inches)}$$

$$P = \frac{2WH''}{12(S'' + k)}$$

$$P \cong \frac{WH''}{6S''} \qquad \text{(assumes k is small)} \qquad (9\text{-}4)$$

2. Compare to total driving capacity,

$$R = \frac{WH}{S}$$

3. Safety Factor = $\frac{R}{P} \cong 6$

Most engineers are not aware of (1) the use for which the ENR formula was originally developed, or (2) the fact that the ENR formula has a built-in factor of safety of 6. Sowers, in his 1979 Introductory Soil Mechanics and Foundation Engineering Text, states the following about the ENR formula: "The ENR formula was derived from observations of the driving of wood piles in sand with free-falling drop hammers. Numerous pile load tests show that the real factor of safety of the formula can be as low as 2/3 and as high as 20. For wood piles driven with free-falling drop hammers and for lightly loaded short piles driven with a steam hammer, the ENR formulas give a crude indication of pile capacity. For other conditions they can be very misleading."

In 1988 the Washington State DOT (WDOT) published a study entitled "Comparison of Methods for Estimating Pile Capacity," Report No. WA-RD-163.1. The study which was based on high quality pile load test data, showed the ENR formula to be the least reliable of the 10 dynamic formulae which were analyzed. More recent studies by FHWA under Demonstration Project 66 have also confirmed the unreliability of the ENR formula, particularly for higher pile loads where actual safety factors are too frequently less than 1.0.

The WDOT study and the FHWA Demonstration Project 66 resulted in both organizations replacing ENR in their specifications with the Gates dynamic formula. However, the formula is usually restricted to piles which have driving capacities less than 600 kips. The Gates formula was originally developed based on correlations with static load test data. The Gates formula, which was modified by FHWA for driving capacity, is shown below:

$$R = 1.75\sqrt{E}\ \mathrm{Log}(10N) - 100 \qquad (9\text{-}5)$$

Where: R = Driving Capacity (kips)
E = Manufacturer rated energy (foot-pounds) at the stroke observed in the field

Log (10N) = Logarithm to the base 10 of the quantity 10 multiplied by N, the number of hammer blows per inch at final penetration (blows per inch)

## 9.6 DYNAMIC ANALYSIS OF PILE DRIVING

An examination of the pile driving process discloses that the concept of a Newtonian impact does not apply. When viewed in slow motion, the ram does not immediately rebound from the pile after impact. The ram transfers force to the pile head over a finite period of time which depends on the properties of the hammer-pile-soil system. A force pulse is created which travels down the pile in a wave shape. The amplitude of the wave will decay due to system damping properties before reaching the pile tip. The force in the wave, which reaches the tip, will "pull" the pile tip into the soil before the wave is reflected back up the pile. After reflection an amount of permanent "set" of the pile tip will remain. This process is crudely shown in (Figure 9-4) for the hammer-pile-soil system.

Each element in the hammer-pile-soil system affects the pile penetration and stresses caused in the pile. A few characteristic effects of each element are discussed below.

1. Hammer

    - Mechanical efficiency determines what percentage of rated energy is transmitted by the ram. Typical values of percent energy transfer for hammers in good repair are 50% for air-steam, 70% for diesel and over 90% for hydraulic.

    - Force wave shape characteristics are different for different hammer types. The shape affects pile stress and pile penetration. Air-steam types generate a wave with high amplitude and a low period. Equivalent diesel hammers generate lower amplitudes but longer periods.

2. Pile and Appurtenances (Cushions, Helmets, Etc.)

    - The stiffness of appurtenances such as the hammer cushion is defined by the cross sectional area times the modulus of elasticity divided by the thickness. The stiffness has a major effect on both blow count and stress transfer to the pile. These elements must not degrade during driving as observed blow count will decrease and pile stresses increase.

    - The cross sectional area of the pile is a major factor in pile driveability. Long piles with small cross sectional areas are so flexible that the hammer energy is absorbed in strain rather than as work to advance the pile against the soil resistance. Pile aids such as mandrels are used to temporarily increase the pile cross section (and stiffness) during driving.

3. Soil

    - Soil strength may be permanently or temporarily changed during driving. Piles being driven into soil containing large percentages of fines may require restrikes to estimate long term capacity due to effects of set-up or relaxation.

    - The damping properties of the soil surrounding the pile can have dramatic effect on observed blow count. An increase in damping decreases driveability. Damping parameters can be estimated by soil type or from basic index test data. Consideration of the dynamic aspects of the field pile driving operation is necessary to relate to the static pile capacity. Foundation designers should routinely consider the potential for dynamic effects such as set-up and include provisions for field observations such as restrikes. In addition, construction control of pile driving should account for basic dynamic parameters which influence blow count and pile stress. Some can be controlled by specification; others require use of a pile wave equation analysis.

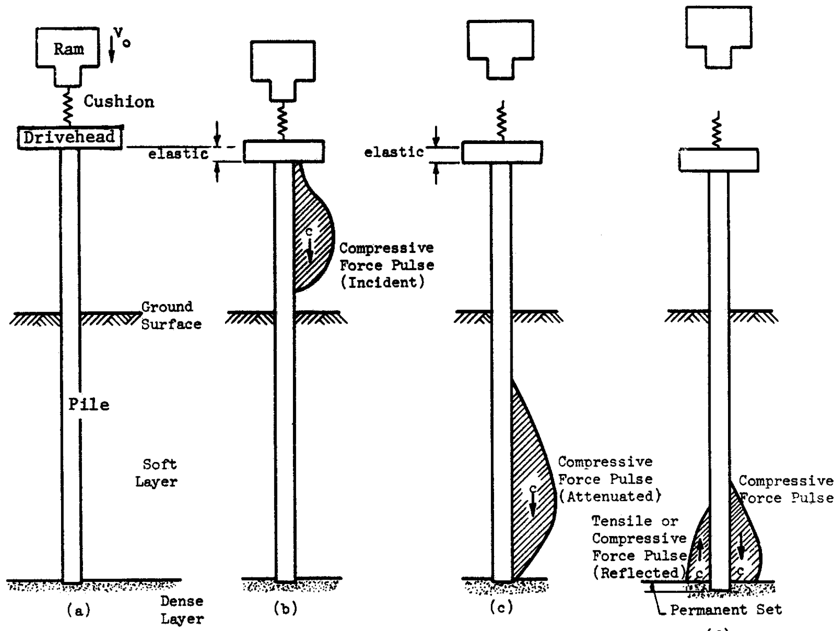



Figure 9-4: Hammer-pile-soil system

## 9.7 WAVE EQUATION ANALYSIS

The wave equation analysis is a computer-based analysis developed from the one dimensional wave equation. The classical wave theory was developed to model wave propagation in a slender rod subjected to an applied force at one end. The pile wave equation uses the wave propagation theory to define the longitudinal wave caused by a hammer impact at the pile top. However, the classical theory must be modified to account for changes in the traveling wave form due to pile and soil properties. Currently the model on which wave equation analysis is based contains a series of masses interconnected by a system of springs and dashpots. These latter elements attenuate the traveling force wave. Pile set is caused by the portion of the force wave which reaches the pile tip. If the traveling wave is completely dissipated by the pile and soil properties prior to reaching the tip, no permanent set will occur and the pile head will rebound. Wave dissipation commonly occurs on projects where either too small a hammer is used or too small a pile cross sectional area is specified for the length being driven.

1. Input to Pile Wave Equation

   Input parameters are required for the hammer, pile (plus appurtenances), and soil. The confidence level which can be assigned to the output is directly related to how well the input parameters are known. The basic input parameters are discussed below.

   Hammer input properties are usually well known and stored in a data file in the wave equation program. In design analysis, hammer types are selected based on the soil resistance to be overcome.

In construction control analysis the contractor submits the hammer type. The major concern in construction is that the hammer is in good working condition as was assumed for the input.

Appurtenance input consists of weight and/or stiffness values. The properties of cushions are especially critical. Only manufactured materials whose properties remain constant during driving can be used with confidence. The actual cushion thickness used in the field must be checked and discrepancies reported so that pile wave analysis can be modified.

Pile length and cross sectional area are major input items. The pile wave analysis cannot predict pile length. This fact is commonly misunderstood by engineers. Pile length is determined by static analysis procedures and then used as input to pile wave analyses. Cross sectional area of the pile is frequently varied in design analyses to determine which section is both driveable and cost effective. Increasing the pile section has the effect of improving driveability as well as reducing pile stresses.

Soil data input requires both an understanding of site specific soil properties and the effects of pile driving on those properties. Dynamic properties such as damping and quake are roughly correlated with soil type. These properties are best determined by experienced geotechnical engineers. The driving soil resistance and its distribution are determined from the static analysis. Remember that the driving soil resistance may be substantially greater than the design load times the safety factor; particularly for scour piles. Also the dynamic effects of pile driving on soil resistance must be considered by an experienced geotechnical engineer to determine set-up or relaxation values for ultimate soil resistance. These dynamic effects are frequently overlooked and can result in large variations between estimated and actual pile lengths.

2. Output Values from Pile Wave Analysis

The results of a pile wave analysis include the predicted blow count, pile stresses, and delivered hammer energy for an assigned driving soil resistance, $R_{ult}$, assuming given hammer, appurtenance, and pile conditions. Remember that each pile wave analysis is for the specific pile length that was input. The summary table of output shown below was generated for a specific site where a pile length of 50 feet was being analyzed.

## WAVE EQUATION SUMMARY

| $R_{ult}$ kips | Blow Count BPF | Stroke (EQ) Ft. | Tensile Stress Ksi | Compressive Stress Ksi | Transfer Energy Ft-Kip |
|---|---|---|---|---|---|
| 35.0 | 7 | 3.27 | -0.73 | 1.68 | 13.6 |
| 80.0 | 16 | 3.27 | -0.32 | 1.71 | 13.6 |
| 140.0 | 30 | 3.27 | -0.20 | 1.73 | 13.0 |
| 160.0 | 35 | 3.27 | -0.14 | 1.73 | 13.0 |
| 195.0 | 49 | 3.27 | -0.00 | 1.75 | 12.8 |
| 225.0 | 63 | 3.27 | 0.0 | 1.96 | 12.7 |
| 280.0 | 119 | 3.27 | 0.0 | 2.34 | 12.6 |
| 350.0 | 841 | 3.27 | 0.0 | 2.75 | 12.5 |

** Note that for each driving resistance ($R_{ult}$), a value of blow count, hammer stroke, tensile stress, compressive stress, and transferred energy has been computed. The data is also commonly shown in graphical form as noted in the following plot.

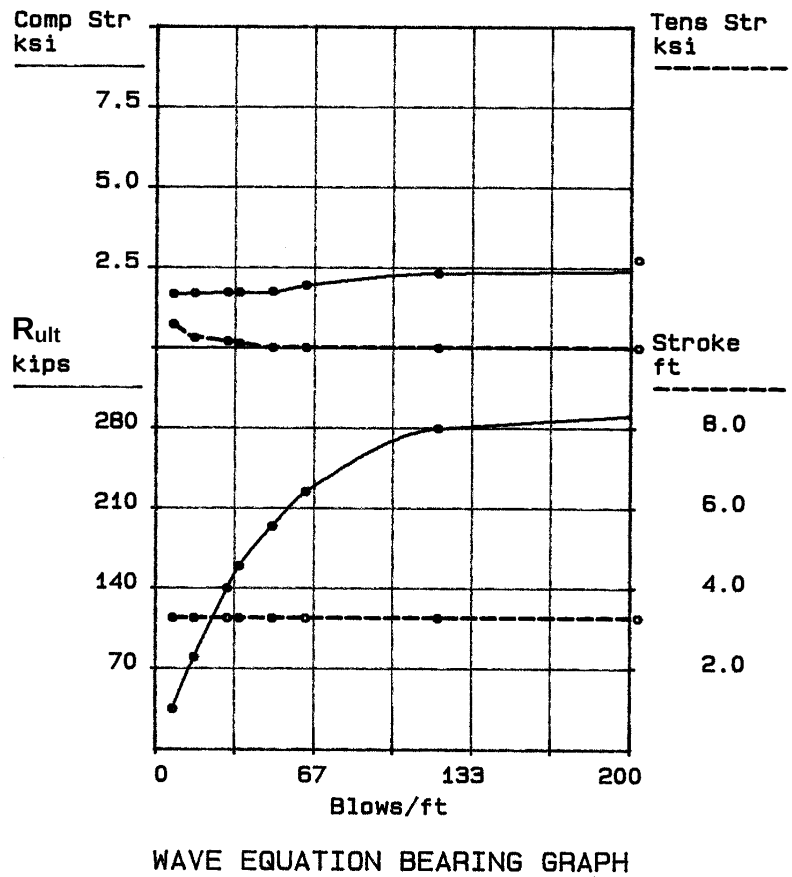



Figure 9-5: Grlweap – Summary of Compressive Stress, Tensile Stress, and Driving Capacity vs. Blow Count

3. Pile Wave Equation Analysis Interpretation

The summary table of output shown in Figure 9-5 contains the predicted relationship between pile hammer blow count and other variables for the situation when the pile is embedded 50 feet in the ground. Therefore, the data in the table is interpreted in the field by comparing the measured blow count at a pile penetration of 50 feet with the data in the summary table i.e., when the pile reaches 50 feet, if the blow count is 49, the driving capacity is 195 kips, the stroke 3.27', the tensile stress zero ksi, the compressive stress 1.75 ksi, and transferred energy 12.8 ft-kips. If the blow count had been 63 the driving capacity would have been predicted to be 225 kips, etc.

Note that this summary table is for an air-steam hammer and the stroke is constant for all blow counts. Diesel hammers operate at different strokes depending on the pile-soil properties. A pile wave summary table for a diesel hammer will display a predicted combination of blow count and stroke which is necessary to achieve the driving capacity. In fact, there are numerous combinations of blow count and stroke which correspond to a particular driving capacity. These combinations may be computed and plotted for a selected driving capacity using the constant capacity output option of the wave equation. A typical plot of diesel hammer stroke versus blow count is shown in Figure 9-6 for a constant capacity of 240 kips.

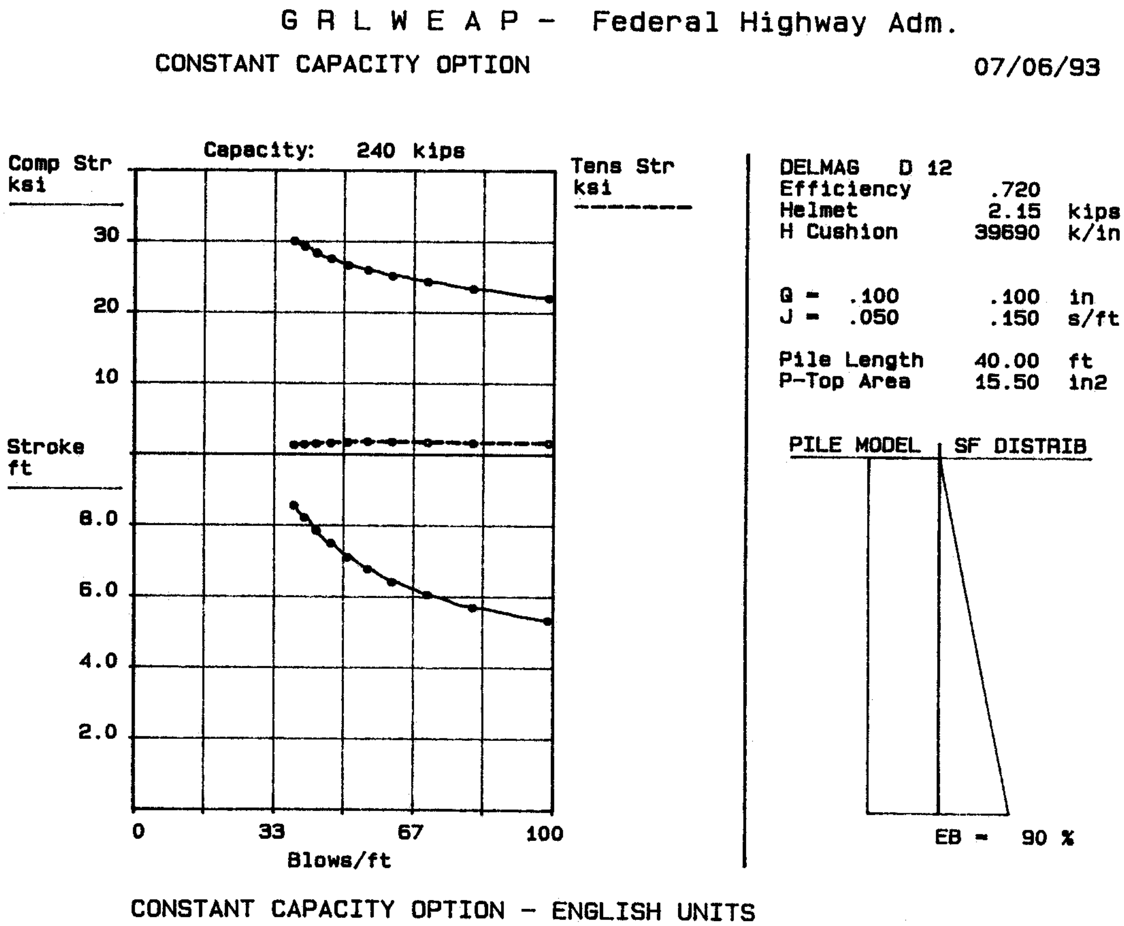



Figure 9-6: Graph of stroke versus blow count for a constant pile capacity

The stresses predicted by the wave analysis should be compared to safe stress levels. This comparison is usually performed for the tensile and compressive stress shown at the computed driving resistance for the estimated pile length. FHWA publication RD 83-059, "Allowable Stresses in Piles", presents detailed information on dynamic loading of piles. From that report FHWA has developed the following limiting stress levels to prevent pile damage during driving.

| Pile Type | Allowable Driving Stress |
|---|---|
| Steel | 0.9 Fy |
| Concrete | (0.85 F′c – effective prestress) in compression |
| | (3 √F′c + effective prestress) in tension |
| Timber | 3 F′a (not to exceed 3000 psi) |

Where: Fy = Yield strength of steel
F′c = 28 day concrete cylinder strength
F′a = allowable compressive stress of timber including allowance for treatment effects

The last operation in pile design is to insure that the pile can be driven to the estimated length without damage. For this purpose a trial wave equation analysis is done with an appropriately sized hammer. Figure 9-7 can be used to choose a reasonable hammer for wave analysis.

In general Figure 9-7 hammer energies are lower than the optimum energy necessary to drive the appropriate pile cross section. Judgement should be used in selecting the hammer size. If initial wave equations yield high blow counts and low stresses the hammer size should be increased. ~~In design wave equation analysis the designer should determine if a reasonable range of hammer energies can drive the proposed pile section without exceeding both the allowable driving stresses (above) and a reasonable range of hammer blows, ie 30 to 144 for friction piles and higher blows of short duration for end bearing piles.~~

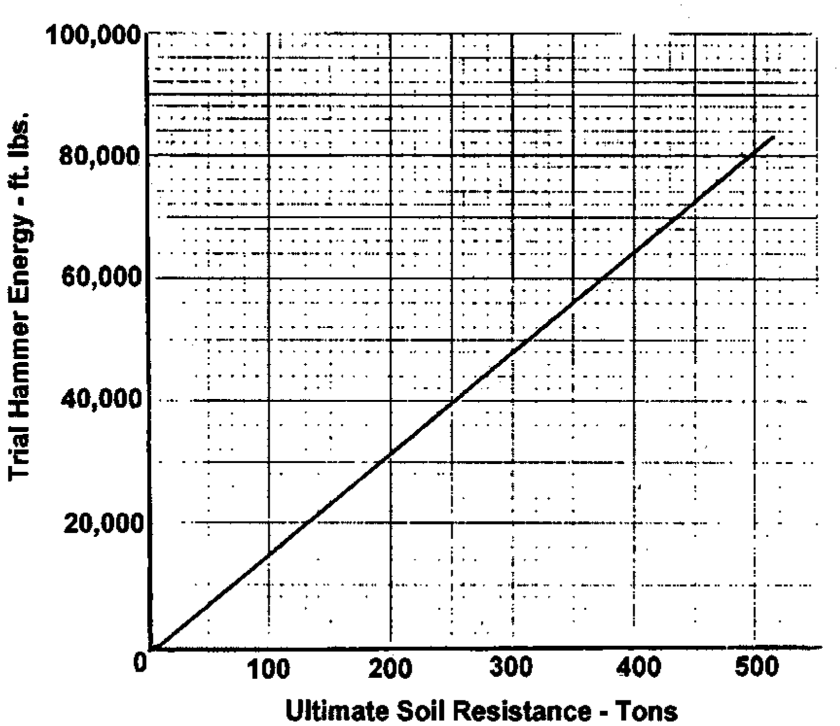



Figure 9-7: Suggested trial hammer energy for wave equation analysis

**Example 9-1:** Determine If The 14″ Square Concrete Pile Can Be Driven To A Driving Capacity Of 225 kips By Using The Wave Equation Output Summary. Assume The Concrete Compressive Strength Is 4000 psi And The Pile Prestress Force Is 700 psi.

## WAVE EQUATION OUTPUT SUMMARY

| $R_{ult}$ kips | Blow Count BPF | Stroke (EQ) Ft. | Tensile Stress Ksi | Compressive Stress Ksi | Transfer Energy Ft-Kip |
|---|---|---|---|---|---|
| 35.0 | 7 | 3.27 | -0.73 | 1.68 | 13.6 |
| 80.0 | 16 | 3.27 | -0.32 | 1.71 | 13.6 |
| 140.0 | 30 | 3.27 | -0.20 | 1.73 | 13.0 |
| 160.0 | 35 | 3.27 | -0.14 | 1.73 | 13.0 |
| 195.0 | 49 | 3.27 | -0.00 | 1.75 | 12.8 |
| 225.0 | 63 | 3.27 | 0.0 | 1.96 | 12.7 |
| 280.0 | 119 | 3.27 | 0.0 | 2.34 | 12.6 |
| 350.0 | 841 | 3.27 | 0.0 | 2.75 | 12.5 |

**Solution:**

Acceptable driveability depends on achieving the hammer blows between 30 and 144 at the driving capacity, and assuming that the allowable compressive and tensile driving stress are not exceeded.

1. At $R_{ult}$ = 225 kips, blow count = 63 O.K. between 30 and 144

2. For concrete piles, the allowable driving stresses are:
   - Compressive stress allowed = 0.85 F′c – prestress = 3400 – 700 = 2700 psi, actual maximum compressive stress up to 225 kips from wave equation output summary is 1.96 ksi or 1960 psi ≤ 2700 psi allowed value. O.K.
   - Tensile stress allowed = 3√F′c + prestress = 190 + 760 = 890 psi, actual maximum tensile stress up to 225 kips from wave equation output summary is 0.730 ksi or 730 psi ≤ 890 psi allowed value. O.K.

Therefore driveability is acceptable.

## 9.8 PILE CONSTRUCTION CONTROL CONSIDERATIONS

The approval of a contractor's driving equipment is an example of design and field coordination. The recommended procedure uses the results of a wave equation analysis to determine if the contractor's equipment is adequate to drive the pile to the estimated length without pile damage. The steps in this procedure are as follows:

1. The pile specifications should include a statement similar to:

   "All pile driving equipment to be furnished by the contractor shall be subject to the approval of the engineer. Prerequisite to such approval, the contractor shall submit the following:

   a. A completed pile and driving equipment data form (Figure 9-8) for each hammer proposed for the project.

   b. A wave equation analysis performed by a professional engineer for each proposed hammer to the soil resistance value listed on the plans.

   Contractor notification of acceptance or rejection of the hammer will be made within 14 days of receipt of the data form and wave equation analysis."

2. The designer should also receive a copy of the data form and the wave equation results. An independent wave equation should be done to verify the submitted results and to establish driving criteria for the piles. In approving equipment, the designer enters the pile driving information form with the soil resistance to be overcome during driving and establishes if a reasonable number of blows per foot are required to attain that resistance. For friction piles, 30-144 blows per foot are considered reasonable. Higher blow counts can be permitted for end bearing piles since the duration of high blow counts is short. Then the stresses at that blow count are checked to determine if the values are below the allowable driving stress of the pile material. If these items are satisfied, the equipment can be approved and the information sent to the field engineer. The wave equation results may be transmitted to the field with a recommendation to reject or approve the hammer. If

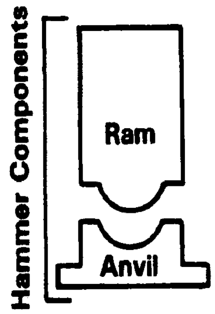
Contract No.: ____________________ Structure Name and/or No.: ____________________

Project: ____________________ ____________________

____________________ Pile Driving Contractor or Subcontractor: ____________

County: ____________________ ____________________

(Piles driven by)




Hammer

Manufacturer: ____________ Model: ____________

Type: ____________ Serial No.: ____________

Rated Energy: ____________ at ____________ Length of Stroke

Modifications: ____________________

____________________

____________________

____________________

____________________

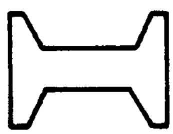
Hammer Cushion

Material: ____________________

Thickness ____________ Area: ____________

Modulus of Elasticity – E ____________ (P.S.I.)

Coefficient of Restitution-e ____________



Drive Head

- Helmet
- Bonnet
- Anvil Block
- Pile Cap

– Weight: ____________________

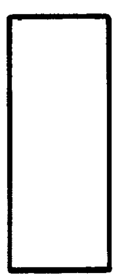
Pile Cushion

Cushion Material: ____________________

Thickness: ____________ Area: ____________

Modulus of Elasticity – E ____________ (P.S.I.)

Coefficient of Restitution ____________



Pile

Pile Type: ____________________

Length (In Leads) – ____________________

Weight/ft. ____________________

Wall Thickness: ____________ Taper: ____________

Cross Sectional Area ____________ $in^2$

Design Pile Capacity: ____________ (Tons)

Description of Splice: ____________________

____________________

Tip Treatment Description: ____________________

____________________

Distribution
One Copy Each To:

- ☐ State Bridge Engineer
- ☐ State Soils Engineer
- ☐ District Engineer
- ☐ Resident Engineer

Note: If mandrel is used to drive the pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: ____________________ Date: ____________

Figure 9-8: Pile and Driving Equipment Data Form the hammer is recommended for approval, the transmittal will contain pile driving criteria.

3. The procedure for hammers proposed for substitution during the contract is the same.

During production operations, the engineer will check to see if the necessary blow count is attained at the estimated length shown on the pile driving information form. The resistance is generally acceptable if the blow count is within 10 percent of that expected, or if the expected blow count is achieved within 5 feet of the estimated length. The field engineer should be aware that blow counts higher than expected will cause an increase in pile stress the correlation of blow count and stress is shown on the information form. If necessary an upper blow count limit may need to be established to prevent damage.

Most important, if either radically different blow counts (higher or lower) or damage are observed at the estimated length, the foundation engineer should be contacted immediately. The phone number of the foundation engineer should be on the information form.

All engineers should realize that pile driving is not by any means an exact science and actual blow counts may be expected to vary somewhat even in the same footing. The objective of construction control of pile driving is to insure that the pile is capable of safely carrying the design load. This means that the driven pile is not damaged and enough soil resistance is mobilized for support. Both these items can be checked from the pile wave analysis output.

The pile wave equation analysis is a big improvement over dynamic formulas because the variables of pile length and flexibility are accounted for in addition to the variations in the contractor's driving system and the project soils. A wave equation program entitled "WEAP" was developed for FHWA in 1975 and updated periodically thru 1987 when FHWA development activities ceased. However, since 1988 numerous technical changes and programming improvements have been made periodically to the WEAP model by GRL and Associates Inc. The resulting proprietary program, GRLWEAP, has been accepted by public agencies for use on a variety of public projects since 1988.

The use of wave equation in construction control provides the engineer with a prediction of the behavior of the driven piles during installation. While this prediction is superior to previous methods of estimating driveability, the optimal method of determining pile driveability is to obtain dynamic measurments during pile installation. Dynamic test methods commonly employ accelerometers and strain gages, attached to the pile during driving, to measure real time strains and accelerations produced during the driving process. Field computers use these measurements to output information, which the inspector can use to:

- Monitor hammer and driving system performance,
- Evaluate driving stresses and pile integrity, and,
- Verify pile capacity

Additional details of the dynamic test procedure are shown in section 9.13.

## 9.9 DEEP FOUNDATION SPECIFICATIONS

### 9.9.1 Pile Specification

Early pile specifications placed the major responsibilities for pile capacity determination on the field staff. Little analysis was done in design to provide accurate estimates of the required pile length to safety support the design load. No design analyses were done to account either for the actual soil resistance to be overcome to drive the pile to the estimated length or the stresses generated in the pile during driving. Specifications frequently placed the responsibility of determining what pile length to order on the contractor. Delays for reordering additional lengths or splices to reach final tip were considered incidental to the price bid for the item. The result was higher bid prices to account for the risks involved with the pile item.

At present, procedures, equipment and analysis methods exist to permit the designer to accurately establish pile length and section for any driving condition. Basic foundation design procedures are routinely followed by nearly all public agencies. Yet much of the available design information is neither reflected in the pile specification of the agency nor utilized by the construction staff. Many agencies perform detailed static analyses to determine pile length, but control the pile length actually installed in the field with the Engineering News Formula which is known to be the least accurate and least reliable of all dynamic formulas. Improvements are required in the pile specifications to permit the cost effective use of the state-of-the-art pile techniques. A model driven pile specification is included in the FHWA Geotechnical Engineering Notebook which incorporates the improvements listed below.

1. Ordered Length Replaces Estimated Length - Public highway agencies should assume responsibility for determining and placing in the contract documents the pile length necessary to safely support the design load. Extra costs associated with overruns or underruns due to inaccurate length determination should not be borne by the contractor. The concept of a fair pile specification is based on the highway agency performing adequate subsurface work and design analyses to rationally establish pile lengths during the design phase.

2. Driving Capacity Replaces Design Load - Installation of piling to a predetermined length involves overcoming the design soil resistance multiplied by the safety factor in the bearing layer plus the resistance in any overlying layers unsuited for bearing. The use of procedures involving only design load, such as the Engineering News Formula, should be replaced with ultimate load based methods. The ultimate load to be achieved should be based on both the actual resistance to be overcome to reach ordered tip and the confidence in the method of construction control to be used. Ultimate values are now required for load-resistance factor design procedures adopted by AASHTO.

3. Increased Emphasis on Approval of Driving Equipment - The use of properly sized pile driving equipment will practically insure a successful installation of properly designed piles. Conversely, improperly sized pile driving equipment insures a pile project fraught with problems regardless of how well the pile design was done; to small a hammer results in extremely difficult, time consuming driving; too large a hammer results in pile damage. Fair pile specifications should place great emphasis on a formal approval procedure for the hammer and hammer appurtenances. This approval procedure is the most significant improvement which could be made to current specifications.

4. Field Control of Pile Capacity by Wave Equation and Dynamic Pile Testing to Replace Engineering News Formula - Current good piling practice includes the use of the wave equation and dynamic

pile testing in place of dynamic formula to monitor pile driving for all projects. In particular, continued use of the Engineering News Formula can only result in unreliable, costly pile foundations. Highway agencies need to utilize modern methods both in design and construction control of pile foundations. The wave equation is designed to use driving resistance, basic soil properties and calculated pile lengths in conjunction with driving equipment characteristics to produce both the necessary hammer blow count for the desired load and the maximum pile stress to be encountered during driving. Dynamic pile testing provides a quick, reliable field test alternate to static load testing as well as a supplement to pile wave equation analysis.

5. Separation of Payment into Fixed and Variable Cost Items to Replace Lump Sum Items - Fair compensation for work performed in pile driving can only be accomplished by recognizing and providing bid items for those contract costs which are fixed and those contract costs which are variable. Some payment methods used by highway agencies involve lumping fixed and variable costs into a single item. Such lump sum items with variable contingencies are recognized as high risks items by contractors whom, to avoid a monetary loss, increase the price bid to cover the risk. An example of this situation is lumping the cost of pile points into the per foot cost of the pile. Pile points are a fixed cost item. However, when lumped with pile length, the pile point becomes a variable cost item, i.e., the contractor breaks down the point cost to a per foot cost based on estimated length. If an overrun in length occurs the price paid to the contractor for the point increases; conversely an underrun results in an underpayment for the pile point cost.

### 9.9.2 Drilled Shaft Specification

Construction control of drilled shaft work requires that the criteria for field measurements and tests be clearly outlined in the construction specification. However, the most important item to check for a successful project is the qualifications of the drilled shaft contractor. Drilled shaft construction is a specialty item. Specifications must include a procedure for establishing that the contractor possesses both proper tools and expertise to install the size of drilled shaft designed for the project. Projects involving difficult drilling conditions, large diameter (>8') shafts, non-redundant shaft designs, or over-water shaft installation require special expertise. Specifications should require either submittals of qualifications at the time of bid or an on-site demonstration of contractor abilities by constructing to specification a trial drilled shaft previous to installing production shafts.

The specification should communicate specific construction control items which directly relate to the shaft design, i.e., construction requirements for end bearing shafts differ from friction shafts. A discussion follows of general items to be included in the specification. A model drilled shaft specification is included in the FHWA Geotechnical Engineering Notebook.

1. Construction Method - The construction methods to be permitted on a specific project are directly related to the method of load transfer assumed in the project design. The type of drilling method, presence of permanent casing, and clean out procedure all affect the drilled shaft load transfer behavior in skin friction and end bearing. For instance, the permanent casing method cannot be permitted in subsurface deposits which were designed for full mobilization of shaft skin friction with the soil.

   Fortunately, numerous combinations of equipment and procedures are commonly available to permit successful installation of drilled shafts for any stated design criteria. Specifications should not needlessly restrain contractors in their choice of tools, equipment or construction methods. The key to cost effective projects is permitting flexibility in contractor operations to achieve the design intent; particularly at sites where variable subsurface conditions are expected.

Quality of the end product is monitored and controlled by including explicit definitions and controls in the following areas: installation plan, tolerances, acceptance and rejection criteria, and project documentation. The success of specification relies heavily on responsible and knowledgeable inspection, and experienced drilled shaft contractors. Even the most conservative design can result in problems if: the specified construction procedure is inappropriate for the project conditions, the inspection is not effective, or the contractor is poorly equipped or inexperienced with drilled shaft construction.

2. Drilling Slurry - Drilling slurry is an effective method of stabilizing drilled shaft excavations until either a casing has been installed or concrete has been placed. The properties of drilling slurry should be both monitored and controlled prior to and during the drilling, and prior to concrete placement. Primary concerns connected to slurry use are: the shape of the borehole be maintained during the excavation and concrete placement; the slurry does not weaken the bond between the concrete and both the natural soil and rebar; all of the slurry is displaced from the borehole by the rising column of fresh concrete; and any sediment carried by the slurry is not deposited in the borehole.

   The engineer's concerns regarding the behavior and effectiveness of slurry projects can be satisfied by appropriate specification requirements. These requirements include: specifying a suitable range of slurry properties both prior to and during excavation and prior to concreting; performing slurry inspection tests; and construction of preproduction trial shafts by the slurry method.

3. Payment for Shaft Excavation - The ability of a contractor to excavate a particular strata depends on the type, size, and condition of the contractor's equipment, as well as, the skill of the equipment operators. Two alternate methods of payment for shaft excavation can be specified; unclassified payment and classified payment. Both methods require separate compensation for obstruction removal. A single unclassified excavation item alternate was included primarily to avoid ambiguous, unfair and impractical definitions of soil and rock excavation which have been used by some agencies. The engineer's concern regarding high contingency bids, when using this method, can be satisfied by performing an adequate site investigation and making this information available to bidders.

   A. Unclassified Payment

      Unclassified payment is appropriate at sites where a comprehensive exploration program has been completed specifically for the drilled shaft foundation. Such a program should include a full size inspection shaft in representative subsurface areas and a test boring to beyond the anticipated shaft depth at the following intervals:

      i. Non-Redundant (Single) Shaft Foundations

         One boring per shaft;

      ii. Redundant (Multiple) Shaft Foundations

| Shaft Diameter | Guideline Boring Requirement |
|---|---|
| 72" or greater | 1 boring per shaft |
| 48" - 72" | 1 boring per 2 shafts |
| less than 48" | 1 boring per 4 shafts |

The boring logs and inspection shaft logs should contain specific information about equipment used and rate of penetration in addition to soil, rock, obstructions and water conditions. All the aforementioned information should be made available to bidders as part of the contract documents.

B. Classified Payment

Separate items for classified payment should be used for all other projects and defined in terms of standard excavation and special excavation. Standard excavation includes hole advancement with conventional augers fitted with either rock or soil teeth, drilling buckets, and/or underreaming tools. Special excavation is paid when the hole cannot be advanced with conventional tools. Hole advancement under special excavation requires special rock augers, core barrels, air tools, blasting or other methods of hand excavation. All earth seams, rock fragments or voids which are encountered after special drilling commences are paid as special excavation. Obstructions which require unconventional excavation techniques are not considered special excavation for payment but paid under a separate item.

4. Special Bidding Requirement - Drilled shaft costs are controlled by the character of the subsurface materials encountered during excavation. No drilled shaft contractor should be permitted to either bid drilled shaft work or act as a subcontractor to a bidder unless he has: visited the site, inspected soil and rock samples (if made available in the contract documents by the agency) and received the subsurface information made available in the contract documents.

## 9.10 DEEP FOUNDATION LOAD TESTS

A static load test is conducted to measure the response of a deep foundation under applied load. Conventional static load test types include axial compressive, axial tensile and lateral load testing. The cost and engineering time associated with a load testing program should be justified by a thorough engineering analysis and foundation investigation. Load tests are possible on either single elements or groups but due to cost considerations only single element tests are generally performed on production projects. The FHWA has published the results of pile group load tests in sands and clays in FHWA TS-87-221 and 222.

Static load tests provide the best means of determining deep foundation capacity and if properly designed, implemented and evaluated, should pay for themselves on most projects. Depending on availability of time and on cost considerations, the load testing program may be included either in the design or in the construction phase. Dynamic load tests, performed in conjunction with static load tests, greatly increase the cost-effectiveness of a pile load test program and should be specified whenever piles installed by impact driving are load tested.

Many different procedures have been proposed for conducting load tests. The main differences are in the selection of loading systems, instrumentation requirements, magnitude and duration of load increments, and interpretation of results. Some innovative test procedures which are potentially applicable to piles and drilled shafts include the Osterberg load cell and high strain dynamic testing such as the Statnamic test. The Osterberg procedure involves installation of a non-retrievable hydraulic jack at the pile or shaft base. The jack reacts against the larger of the base or skin resistance to cause a failure condition of the weaker resistance. High strain tests involve the use of heavy drop weights or explosive devices (the Statnamic procedure) to create strain and acceleration data which are used to predict capacity.

The purpose of load testing is:

- To develop criteria to be used for the design and installation of the pile foundation, or

- To prove the adequacy of the pile-soil system for the proposed pile design load.

### 9.10.1 Prerequisites for Load Testing

Load testing is not a substitute for an adequate foundation investigation program. In the planning stage of any load test program, the following will be required:

- Adequate subsurface exploration.

- Well-defined subsurface profile.

- Adequate soil/rock testing to determine engineering properties.

- Static analysis results to rationally select foundation type and length, as well as the load test site(s).

### 9.10.2 Advantages of Load Testing

Load testing offers several advantages:

- Allows a more "rational" design. The load transfer can be determined much more reliably by applying a test load to a foundation element than from the results of laboratory tests or based on assumptions.

- Allows use of lower factor of safety. Many foundations are designed using a factor of safety of 3. Testing allows the engineer to use a lower factor of safety which translates into cost savings.

- Improved knowledge regarding load transfer has the potential of permitting an increase in the design load and a reduction in the foundation number or length (for friction elements) with a corresponding savings in foundation costs.

- Verifies that the design load can be attained at selected tip elevation.

The reasons often cited for not load testing include:
- Costs involved.

- Delays to contractor if done as part of construction contract.

- Delay of project if done in the design phase.

The cost of performing a load test should always be weighed against the benefits to be obtained. A load test costing $100,000 could be considered inexpensive if cost savings in the millions resulted. Delay of a project during the design or construction phase is most likely to occur in those instances where the decision to perform load tests is made at the last minute. The need for design phase load tests should be addressed in the early stages of the design phase, and construction phase load tests should be clearly specified in the contract documents. In this way, the load tests are incorporated into the schedules and unforeseen delays are minimized.

### 9.10.3 When to Load Test

The decision whether or not to initiate a load test program on a particular project will be influenced by several factors. The following criteria can be used to assess when load testing can be effectively utilized:

- When the potential for substantial cost savings is readily apparent. This is often the case on large projects, either to determine whether friction pile lengths can be reduced, or whether allowable pile stresses can be increased for end-bearing foundations.

- When safe load carrying capacity is in doubt, due to limitations of an engineer's experience or unusual site or project conditions.

- When soil or rock conditions vary considerably from one portion of a project or another.

- When the design load is significantly higher than typical design loads.

- When time related pile-soil capacity changes are anticipated (i.e., setup or relaxation).

- When using precast concrete friction piles - so that piles can be cast long enough to avoid costly and time consuming splicing during construction.

- When new, unproven pile types and/or pile installation methods are utilized.

- When existing foundations will be utilized to support a new structure carrying heavier loads.

- When a reliable assessment of uplift resistance for lateral behavior is important.

- When, during construction, the load carrying capacity of a pile by hammer formula or dynamic analysis differs from the estimated ultimate load at the anticipated tip elevation (for example, H-piles that "run" when driven into loose to medium dense sands and gravels).

### 9.10.4 Effective Use of Load Tests

1. During Design

   On major projects, the benefit to construction of conducting a load test program in the design phase should be considered. The subsurface profile must be adequately defined to determine the optimal number and locations of load tests as well as the area over which each test can be considered representative for driving of production elements. A design phase static load test program will require highway agencies to prepare and let a construction contract. The unit cost per test will be significantly higher than for tests performed during construction (particularly if over-water testing is involved), due to the mobilization of men, materials and equipment to install a small number of piles. For maximum benefit, the design load test program should be completed at least a year before project advertisement to permit foundation and structural engineers to optimize final design.

   Design phase load tests offer several advantages:

   - Allow load testing of alternate foundation types and selection of most economical foundation.

   - Installation information can be made available to bidders - this should reduce their bid "contingency,"

   - Greatly reduce potential for claims arising from pile driving or shaft installation problems, especially for piles which are difficult to splice.

   - Maximize cost savings for foundations (e.g., permit lower factor of safety, permit changes in design load and number of elements, reduce number of orders-on-contract).

2. During Construction

   Typically, the primary purpose of load tests performed during construction is to verify that the design load does not exceed allowable capacity (proof testing), particularly if set-up or relaxation is anticipated. For drilled shaft and piles installed other than by driving with an impact hammer (e.g., vibrated or auger cast), load tests during construction can be used to confirm that both the soil and the structural foundation element can safety sustain the design load.

   Construction phase load tests are also commonly used to determine final tip elevation of production piles after test drive (indicator) piles are evaluated at estimated length.

3. Limitation of Load Tests

   A load test performed on a single pile does not:

   - Account for long-term settlement

   - Take into account downdrag from settling soils

   - Take into account the effect of group action

   - Eliminate the need for an adequate foundation investigation.

The above must be considered when using load test results to design or analyze deep foundations.

## 9.11 QUICK LOAD TEST METHOD FOR STATIC TESTING

The "Quick Load Test Method" is the recommended method for load testing of piles and drilled shafts on highway projects. The quick load test, originally developed by the Texas Highway Department, is allowed as an optional load test procedure by ASTM D-1143. Basically, this method requires that load be applied in increments of 10 to 15 percent of the design load with load, gross settlement, and other pertinent data recorded immediately before and after the addition of each increment of load. After an increment of load is added the load is maintained constant for a time interval of 2-1/2 minutes before the next increment is added.

The Quick Load Test Method offers the following advantages:

- The load test can be performed in 1-2 hours, versus over 100 hours in the standard AASHTO method, with resultant savings in time and money.

- Construction delay to the project caused by load testing is greatly reduced.

- Full-scale load testing on smaller projects is feasible because of reduced time and costs.

- Simplicity of the testing procedure ensures standardization of the test and easy interpretation and utilization of the results.

Similar advantages can be achieved by using the constant rate of penetration (CRP) load test procedure which is described in ASTM D-1143.

### 9.11.1 Factor of Safety - Static Load Test

To obtain the allowable load, the ultimate failure load determined from a load test should be divided by a factor of safety of at least 2.0. Larger factors of safety may be required:

- -For friction piles in clay, where group settlement may control the allowable load.
- Where total settlement that can be tolerated by structure is exceeded.

### 9.11.2 Rule of Thumb for Piles - Cost-Effectiveness of Quick Load Test

It is difficult to decide how large a project has to be before a pile load test is likely to be cost-effective. On projects where friction piles will be used, experience has shown that load tests will typically show that pile lengths can be reduced at least 15 percent versus lengths that would be required by ENR formula. Therefore, this 15 percent pile length reduction, can be used to establish a simple rule of thumb formula to compute the estimated pile footage which the project must have to make the load test cost effective. This formulas is (0.15) (Cost/L.F. of Pile) (X) = Cost of Load Test.

Where X = The minimum estimated pile footage, the project must have before the load test would probably at least pay for itself.

**Example 9-2:**

Assume a project will require 100 ton design load piles. Estimated pile cost is $15/L.F. (furnished and driven). Estimated cost to perform a Quick Load Test is $15,000. How much estimated pile footage must the project contain for a load test to be cost effective?

$$X = \frac{\$15{,}000}{(0.15)(\$15 / \text{L.F.})} = 6{,}700\,\text{L.F. of Pile}$$

Thus, the project would have to contain an estimated 7,000+ lineal feet of piling for load testing to provide a potential cost savings.

### 9.11.3 Load Testing Details

For information and specifications for compressive, tensile, and lateral load testing consult FHWA Publication SA-91-042, "Static Testing of Deep Foundations."

## 9.12 DAVISSON'S LIMIT

The Davisson limit was developed in conjunction with the wave equation analysis of driven piles and is gaining widespread use. It is primarily intended for use with load test results from driven piles tested in accordance with quick methods.

Davisson's limit is defined as the load corresponding to the movement which exceeds the elastic compression of the pile by a value of 0.15 inches plus a factor equal to the diameter of the pile divided by 120. (For example, for the 18 inch diameter pile shown in Figure 9-9: x = 0.15 in. + (18 in. / 120) = 0.3 in.) Piles exceeding 24 inches in diameter require modification of the limit value to the elastic compression plus the diameter divided by 30.

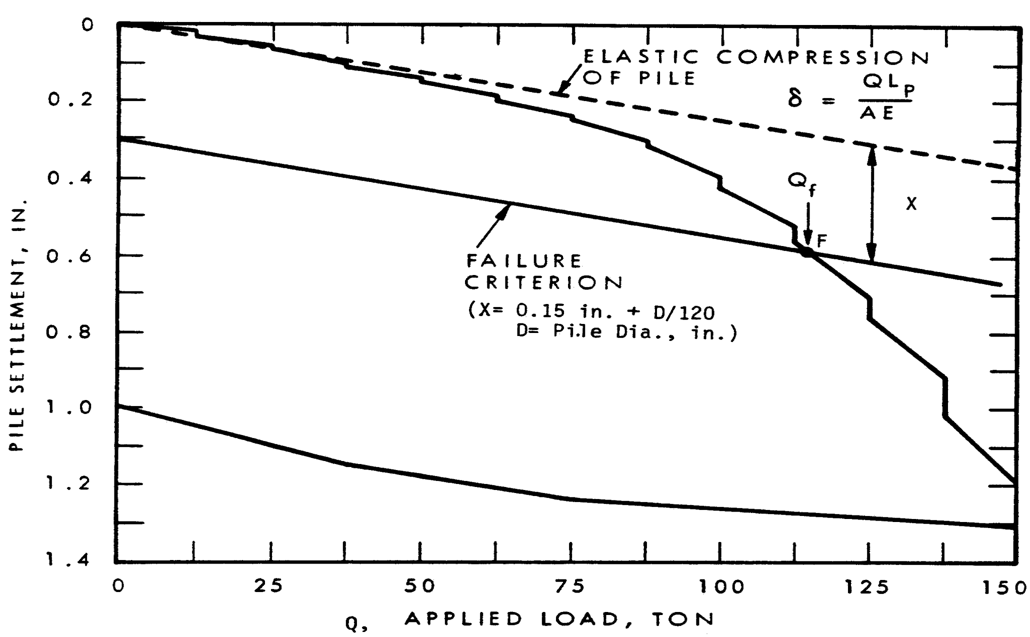



Figure 9-9: Alternate Method Load Test Interpretation (After Davisson)

## 9.13 DYNAMIC PILE LOAD TESTING

Dynamic pile load testing is the estimation of static axial compressive pile capacity from dynamic measurements of pile strain and acceleration. Before the start of testing, two strain transducers and two accelerometers are securely attached to opposite sides of the pile near its top. These gages are connected to the pile analyzer (Figure 9-10). A separate device, such as an oscilloscope may be used to display the data being analyzed and a portable magnetic tape recorder to store the data. These functions are integrated into post 1991 versions of the pile analyzer.

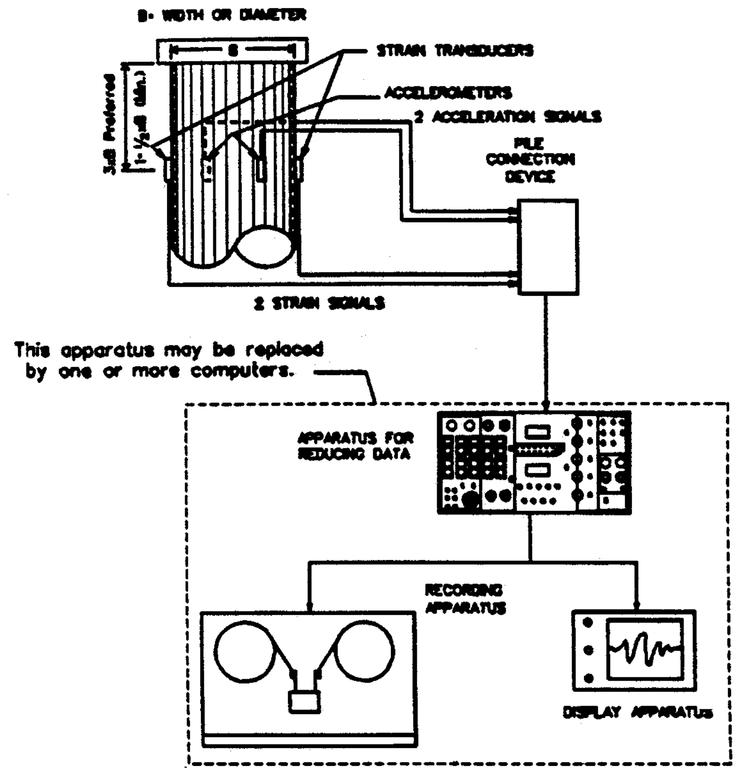
As the pile is struck by a pile hammer, the strains and accelerations detected by the corresponding gages on the pile are converted by the pile analyzer into forces and velocities. The latter quantities are processed to obtain an estimate of the static pile capacity at the time of testing and for pile design. The additional information obtained and displayed includes compressive and tensile stresses in the pile, transferred energy to the pile, and the force and velocity at the top of the pile throughout the duration of the hammer impact. An experienced operator can use this data to evaluate the performance of the pile driving system and the condition of the pile. Usually the results of the dynamic testing are enhanced by performing a computer analysis known as the Case Pile Wave Program (CAPWAP) to verify the correctness of assumed dynamic inputs such as damping.

ASTM D4945-89, Standard Test Method for High-Strain Dynamic Testing of Piles, contains a detailed description of the equipment requirements and test procedure for dynamic pile load testing.

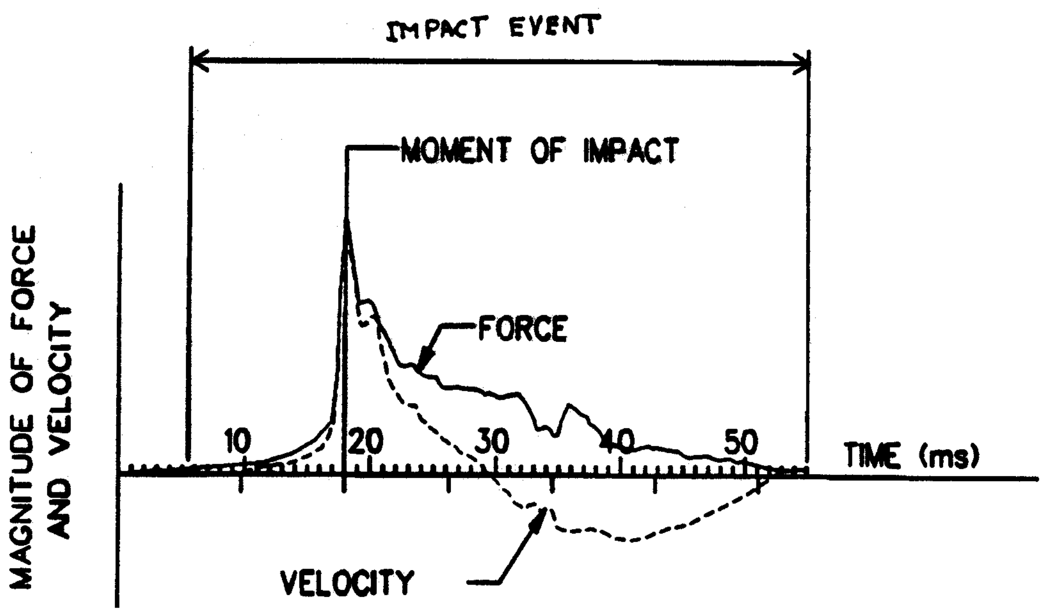
### 9.13.1 Applications

Dynamic pile load testing costs much less and requires less time than static pile load testing. Important information can be obtained regarding the behavior of both the pile-soil system and the pile driving system that is not available from a static pile load test. Consequently, dynamic pile load testing has many applications:

- As a supplement to static pile load testing on major projects, thereby permitting a reduction in the number of static tests.
- On small-scale projects where static pile load tests are difficult to justify economically.
- On projects, such as over-water installations, where full-scale static pile load tests are not feasible.
- To monitor driving stresses and pile integrity.




SCHEMATIC DIAGRAM FOR APPARATUS
DYNAMIC MONITORING OF PILES




Typical Force and Velocity Traces Generated by the Apparatus for Obtaining Dynamic Measurements

Figure 9-10: Schematic Diagram for Apparatus Dynamic Monitoring of Piles

- To evaluate hammer performance.

- To validate wave equation input values.

Dynamic measurements can also be obtained from high strain testing which may be applicable to non-driven piles and drilled shafts. However, consideration should be given to structural damage of the foundation element because the test requires a hammer blow that has sufficient energy to mobilize the capacity of the soil surrounding the foundation element.

### 9.13.2 Interpretation of Results and Correlation with Static Pile Load Tests

The results of dynamic pile load test should be interpreted by an experienced tester who has had the opportunity to observe and evaluate the results from many dynamic load tests and can detect the signs, not always readily apparent, of unusual soil-pile response, pile damage, erratic hammer operation or testing equipment malfunction.

Interpretation of the results of dynamic pile load tests also requires an awareness of the differences in behavior of dynamically and statically loaded piles. Improper correlations of dynamic and static pile load test may be caused by the following:

- Incorrectly assumed soil damping parameters. This source of discrepancy can be minimized by performing a computerized analysis to match measured and computed relationships between force and velocity to determine the most appropriate damping parameter.

- Time-related changes in pile capacity. Depending on soil type and pile characteristics, the capacity of a pile may increase or, less commonly, decrease with time. The principal causes are time-related changes of pore water pressure in the soil. The effects can be assessed by restriking the pile at various time intervals after driving and comparing the capacity against the driving capacity obtained during the initial drive. The pile capacity should be determined during the first few blows of the re-strike. When comparing the results of dynamic testing against those of a static pile load test, at least one dynamic test should be performed after completion of static testing.

- Pile tip displacement during dynamic testing may be inadequate to mobilize full end bearing. Frictional resistance between a pile and the surrounding soil is mobilized at a fraction of the pile movement necessary to mobilize full end bearing resistance. A penetration resistance of 10 blows/inch or higher, may produce insufficient strain in the soil to mobilize full end resistance. This results in an underestimate of the end bearing capacity. For many types of piles, the estimate can be improved by performing a force-velocity match both for the initial drive and for the restrike data. The tip capacity derived from the initial drive is combined with skin resistance from the restrike to obtain the total pile capacity. However, this method may not be applicable for open-ended pipe, H-piles, and precast cylinder piles. In the case of these piles, only the structural area of the pile can mobilize the toe bearing during installation. This is a significantly lower value than what may be experienced in the static load test, since the soil will adhere to the pile with time and create a plug.

  If dynamic pile load tests are performed and interpreted by experienced and knowledgeable testers, the correlation between pile capacities determined from static and dynamic pile load tests is good, i.e., $\pm$ 15 percent. The correlation would not be as good for open-ended and H-piles. However, dynamic load tests on these types of piles would, in general, underestimate the static pile capacity.

## 9.14 ADDITIONAL LOAD TEST METHODS

Two methods of load testing have been introduced in recent years which have been used to varying degrees by highway agencies; the Osterberg Cell and the Statnamic methods. Although the details of each method are beyond the scope of this manual, a short primer follows on each method. For additional details the reader should consult other FHWA publications such as FHWA-HI-97-014, "Design and Construction of Driven Pile Foundations".

### 9.14.1 The Osterberg Cell Method

A recent development for evaluation of deep foundation capacity is the Osterberg Cell test. This test employs a sacrificial pressure jack that is either placed in a open-end foundation element (drilled shaft or pile) or attached to the base of a pile prior to driving. This proprietary device has proven to be a simple and efficient method of applying static load to a deep foundation. As shown below the Osterberg cell uses the ground as a reaction for the test load rather than the common static test that relies on an external reaction system.

The cell has been used at both the base and at intermediate levels in open-end foundation elements. In the case of drilled shafts the cell is commonly attached to the rebar cage and lowered into the hole. In the case of piles, the cell is attached to driven displacement type piles such as closed-end pipe or concrete piles prior to installation. The cell may be installed after driving either open-end pipe piles or mandrel driven piles.

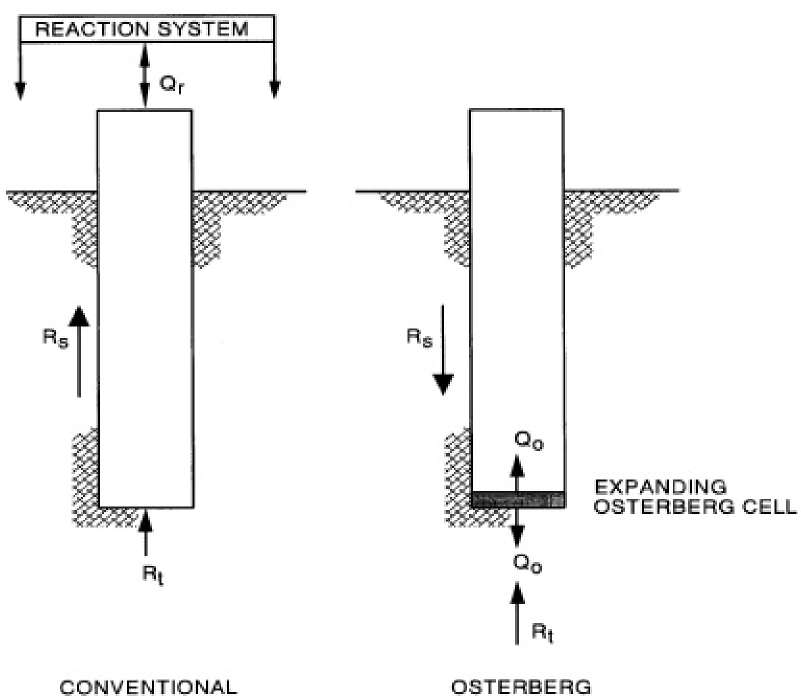



Figure 9-11: Comparison of reaction mechanism between Osterberg Cell and Static Test

As of late 1999, the Osterberg cell is manufactured in a variety of sizes for both open-end installations and driven pile installations. The tables below, which were provided by the American Equipment and Fabricating Corporation, East Providence, RI 02914, contain pertinent dimensions and load capacities for Osterberg cells. Table A refers to cells placed in previously installed foundation elements; Table B refers to driven piles.

Table A – Osterberg Cells (In-place) 1999

| Size | Diameter Inches | Height Inches | Capacity Tons | Weight Pounds |
|---|---|---|---|---|
| 5 | 5.25 | 5.18 | 75 | 32 |
| 9 | 9.00 | 10.75 | 200 | 190 |
| 13 | 13.00 | 11.65 | 400 | 300 |
| 21 | 21.25 | 11.65 | 1200 | 800 |
| 26 | 26.25 | 11.65 | 1800 | 1230 |
| 34 | 34.25 | 12.37 | 3000 | 2015 |

Table B – Osterberg Pile Load Cells 1999

| Capacity - Tons | Size - Inches | Stroke - Inches | Description |
|---|---|---|---|
| 200 | 14 | 6 | Round-Steel pipe |
| 300 | 14 | 6 | Square-Precast Concrete |
| 900 | 18 | 8 | Round-Steel pipe |
| 950 | 30 | 9 | Square-Precast Concrete |

The Osterberg Cell test does have some limitations in that the total failure load of the foundation element is not usually measured; only the failure load of the friction above the cell or the resistance below the cell are measured. Failure can be inferred by extrapolating the non-failure portion of the test in some cases. The Osterberg Cell test has not been standardized by AASHTO or ASTM as of 1999. Additional information on the Osterberg Cell test can be found in FHWA publication FHWA SA-94-035 or at www.loadtest.com

The Osterberg Cell has been used in a variety of soil and rock conditions. The cell has been used to determine the bond stress in rock sockets or in dense glacial tills. In these applications, the use of a caliper device is highly desirable to determine the exact dimensions of the drilled socket. The actual dimensions of holes drilled into intermediate geomaterials can vary from the diameter of the drilling tool due to a variety of geologic factors or drilling considerations. Calipers are available in either mechanical or electronic configurations. In addition, a variety of strain gage devices have been used in conjunction with the Osterberg Cell test to develop a distribution of resistance along the foundation element. Such measurements can also be taken below mid-height cells by extending instrumented rebar below the base of the cell.

### 9.14.2 The Statnamic Test Method

Another recent development for evaluation of foundation load capacity is the Statnamic test method. The Statnamic method is a proprietary method developed by the Berminghammer Foundation Corporation (www.berminghammer.com). A new ASTM draft standard, entitled “Standard Test Method for Piles Under Rapid Axial Compressive Load”, has been proposed but had not been approved as of 1999.

The Statnamic test method uses solid fuel burned within a pressure chamber to create a rapidly increasing pressure between a reaction mass and the top of the foundation element. As the gas pressure increases, the reaction mass is accelerated upward and the foundation element is forced into the ground. After the maximum pressure is achieved, the pressure vents and the element rebounds. A schematic of the test method is shown in Figure 9-12.

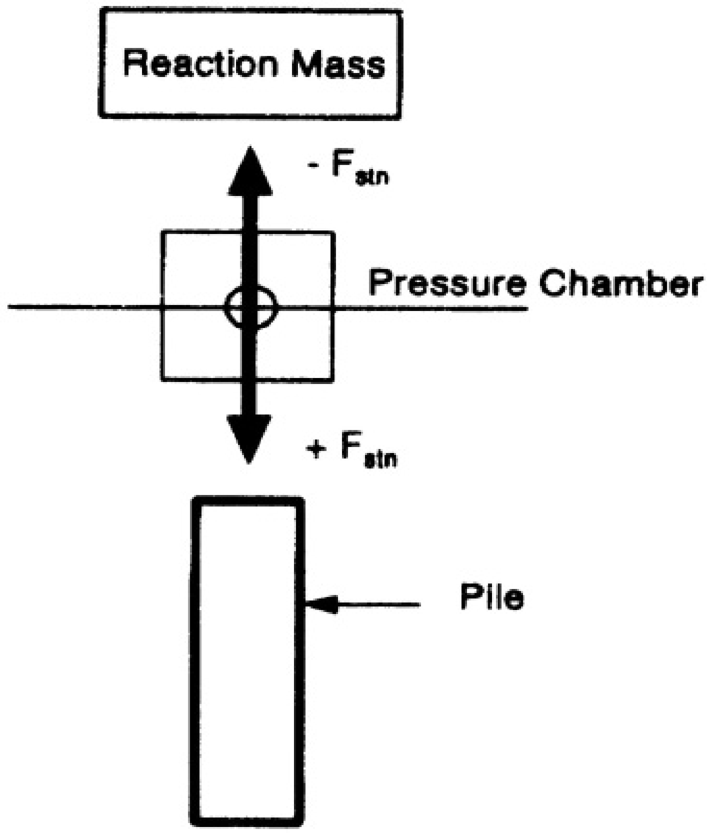



Figure 9-12: Schematic of Statnamic Test Method

Built-in instrumentation (load cell, accelerometers, and laser sensors) is used to measure load, acceleration and displacement. The instruments produce data that permit plots of load and displacement with time to be done in the field. The data is then converted into the familiar plot of load versus displacement shown in Figure 9-13 to permit interpretation of the failure load.

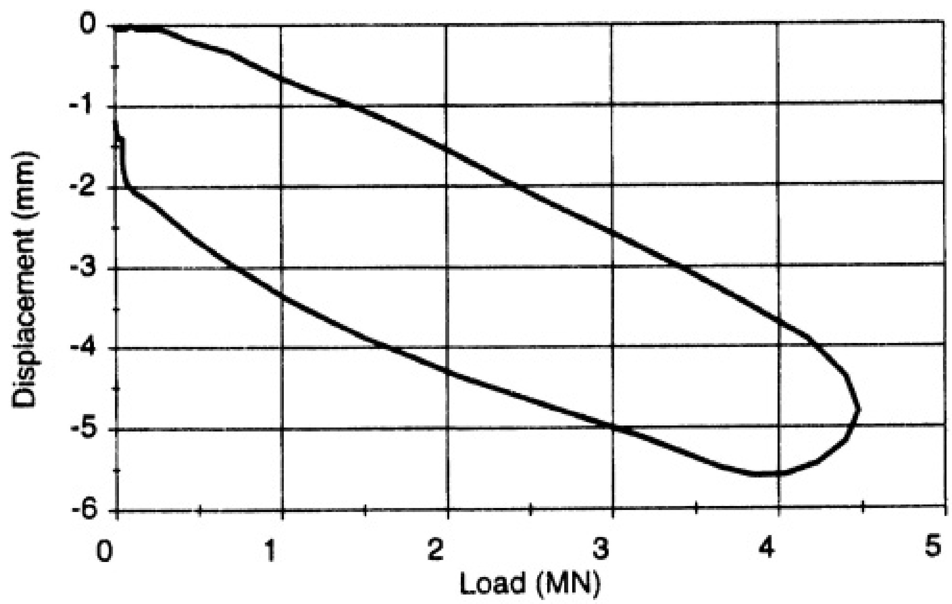



Figure 9-13: Load vs. Displacement plot generated from Statnamic Test

## 9.15 APPLE FREEWAY DESIGN EXAMPLE – WAVE EQUATION ANALYSIS

In this chapter the Apple Freeway Design Example is use to illustrate the wave equation analysis using the GRLWEAP program. The use of GRLWEAP for pile driveability analysis, checking suitability of contractors driving system, and determining pile driving criteria is addressed.

| | |
|---|---|
| Site Exploration | Terrain Reconnaissance<br>Site Inspection<br>Subsurface Borings |
| Basic Soil Properties | Visual Description<br>Classification Tests<br>Soil Profile |
| Laboratory Testing | $P_o$ Diagram<br>Test Request<br>Consolidation Results<br>Strength Results |
| Slope Stability | Design Soil Profile<br>Circular Arc<br>Analysis Sliding Block<br>Analysis Lateral Squeeze |
| Embankment Settlement | Design Soil Profile<br>Settlement<br>Time – Rate<br>Surcharge<br>Vertical Drains |
| Spread Footing Design | Design Soil Profile<br>Pier Bearing Capacity<br>Pier Settlement<br>Abutment Settlement<br>Vertical Drains<br>Surface |
| Pile Design | Design Soil Profile<br>Static Analysis – Pier<br>Pipe Pile<br>H – Pile<br>Static Analysis – abutment<br>Pipe Pile<br>H – Pile<br>Driving Resistance<br>Abutment Lateral Movement |
| → Construction Monitoring | Wave Equation<br>Hammer Approval<br>Embankment<br>Instrumentation |

Apple Freeway Design Example – Construction Monitoring
Exhibit A.

**APPLE FREEWAY WAVE EQUATION ANALYSIS**

**Given:** Using the soil profile and pile driving resistance previously computed (Chapter 8)

**Required:** Complete wave equation analyses using the GRLWEAP program for the following:

- Driveability of the proposed design pile section
- Acceptance of contractors driving system
- Production pile driving criteria

**Solution:**

**Driveability of Proposed Design Pile Section**

Proposed pile section is a 12 $\times$ 84 H – Pile. The maximum driving resistance determined from static analyses is at the east abutment where the total driving resistance including embankment penetration is 689.4 kips. Perform wave equation analysis for the proposed pile section using the maximum driving resistances.

**Step 1: Prepare Wave Equation Input**:

1. Select hammer (IHAMR)

   - Hammer size selected from Figure 9-7 using maximum driving resistance of 689.4 kips (345 tons). Minimum hammer energy = 57,000 ft-lbs.

   - Using GRLWEAP help screen, scan hammer library for hammers with sufficient energies. Select Delmag 30 – 13 hammer which has slightly more energy than required (66,000 ft-lbs) to insure efficient driving.

2. Select uniform or non-uniform pile cross-section along the entire pile (NCROSS)

   - Select non-uniform option (1) because pile point will be used. Pile cross-section described on input screen NCROSS =1 as shown on “page 7” of the GRLINP input printout.

3. Select percent pile skin friction (IPERC)

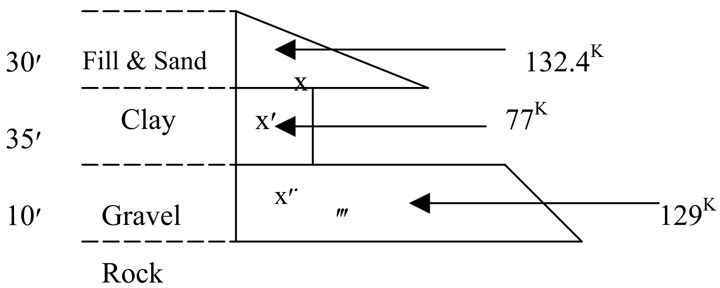
   - From static analysis, (skin friction resistance/total driving resistance) = 338/689 = 49%.

4. Select skin friction distribution (ITYS)

   - Use the actual skin friction distribution determined in the static analysis ultimate driving resistance computation (ITYS = 0). An analysis using the actual skin friction distribution is more realistic and accurate. From static analyses (see next page).

5. Select helmet and hammer cushion information (helmet weight, hammer cushion area, elastic modulus and thickness).
   - Using GRLWEAP help screen, scan hammer library for Delmag Hammers using the proposed

pile section.




$$x = \frac{132.4}{30} \times 2 = 8.83$$

$$x' = \frac{77}{35} = 2.2$$

$$x'' = \frac{5250}{5600}\left(\frac{129}{10}\right) = 12.08$$

$$x''' = \frac{5950}{5600}\left(\frac{129}{10}\right) = 13.72$$

6. Select Pile Top Information (length, x-section area, elastic modulus, specific weight, coefficient of restitution).

   - Using GRLWEAP help screen, obtain the x-sectional area and weight for the proposed HP12×84.

7. Select soil parameters (quake and damping).

   - Using GRLWEAP help screen, select appropriate soil parameters.

8. Select ultimate driving capacities to be analyzed.

   - Input a range of ultimate driving capacities around and including the maximum driving resistance calculated from static analyses (689.4 kips). A range of capacities highlights trends within the graphical plots.

**GRLINP Input Screens for HP12×84 Driven through the Embankment Material – Ultimate Resistance 689.4 kips.**

```
Title:  APPLE FREEWAY H-PILE @ ABUTMENT                             Page:    1

ANALYSIS OPTIONS
     IOUT       IJJ      IHAMR      IOSTR      IFUEL       IPEL
       .          .        13.          .          .          .

ANALYSIS OPTIONS
        N      ISPL     NCROSS     IBEDAM     IPERCS     ISMITH     DMPEXP
        .         .         1.          .        49.          .         .0

ANALYSIS OPTIONS
     ITYS      IPHI      IRSAO       ITER      IDAHA      IMAXT
        .         .          .          .          .          .

HELMET AND HAMMER CUSHION INFORMATION
   Helmet    ┌─────────────── Hammer Cushion ──────────────────────────┐
   Weight       Area   ElasMod Thickness     C.O.R.  RoundOut Stiffness
     2.15     283.50     280.0     2.000       .800     .0100        .0
```

```
Title:  APPLE FREEWAY H-PILE @ ABUT FULL EMBEDMT                    Page:    2

PILE CUSHION INFORMATION
             Elastic                             Round
    Area     Modulus  Thickness      C.o.R.        Out   Stiffness
     .00          .0       .000        .500      .0100          .0

PILE TOP INFORMATION
     Total     X-Sectn  Elastic    Specific                  Round    D.O.A.1    D.O.A.2
    Length        Area  Modulus     Weight       C.o.R.        Out    Slck P2   Stiff P2
     75.00       24.60  30000.0     492.00         .850      .0100        .00        .00

HAMMER OVERRIDE VALUES
                                   Reaction   ComDelay    Comb Exp    Stroke
    Stroke       Effcy   Pressure    Weight    Ign Vol      Coeff    Conv Crit
       .00        .000         .0      .000       .000        .00        .00

SOIL PARAMETERS
    ┌──── Quake ────┐    ┌──── Damping ────┐    ┌────────── Toe No. 2 ──────────┐
      Skin       Toe       Skin        Toe      Quake   Damping  Fraction      Depth
      .100      .100       .050       .150       .000      .000      .000        .00
```

```
Title:  APPLE FREEWAY H-PILE @ ABUTMENT                             Page:    3

ULTIMATE CAPACITIES
Give up to 10 Capacities (5 on first line)
   350.00     550.00     631.80     689.40     740.00

ULTIMATE CAPACITIES (Continued)
(5 on the second line)
      .00        .00        .00        .00        .00
```

**CONTINUE GRLINP INPUT SCREENS FOR HP12x84 (689.4 kips).**

```
Title:  APPLE FREEWAY H-PILE @ ABUTMENT                        Page:

NCROSS=1:    NON-UNIFORM PROFILE, 1ST PILE        ** PILE LENGTH:          75.00
                            Elastic    Specific
    Depth         Area      Modulus     Weight
      .00        24.60      30000.0     492.00
    74.60        24.60      30000.0     492.00
    74.60        42.50      30000.0     492.00
    75.00        42.50      30000.0     492.00
```

75500 PILE POINT (12" H)
AREA = 42.5 in$^2$
HEIGHT = 4.8 in

Title: APPLE FREEWAY H-PILE @ ABUTMENT    Page: 8

ITYS=-1, 0: SKIN FRICTION DISTRIBUTION, 1ST PILE ** PILE LENGTH: 75.00

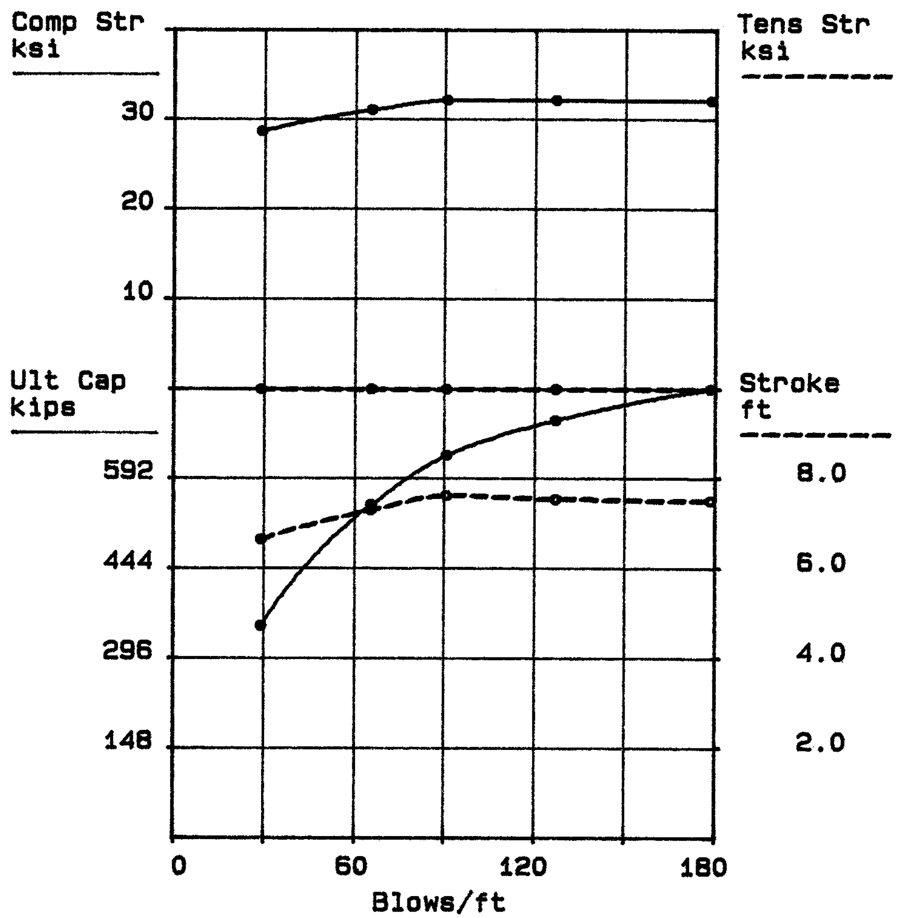
| Depth | Relative Distribn |
|---|---|
| .00 | .000 |
| 30.00 | 8.830 |
| 30.00 | 2.200 |
| 65.00 | 2.200 |
| 65.00 | 12.080 |
| 75.00 | 13.720 |

## SUMMARY OF GRLWEAP RESULTS FOR HP12X84 (DELMAG 30-13)

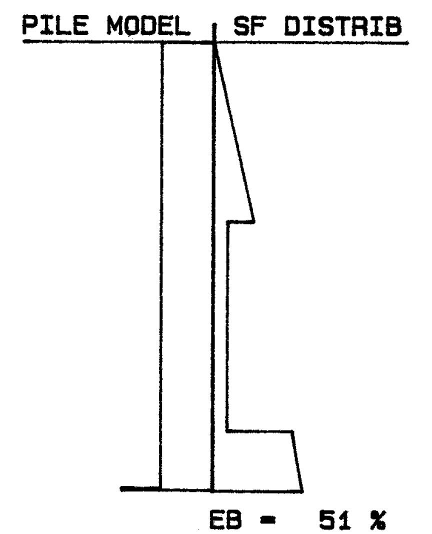
| Rut (kips) | Bl Ct (bpf) | Stroke (ft) down | up | min Str (ksi) | max Str (ksi) | ENTHRU (kip-ft) | Bl Rt (b/min) |
|---|---|---|---|---|---|---|---|
| 350.0 | 32.9 | 6.83 | 6.70 | .00( | 29.09( | 25.9 | 45.3 |
| 550.0 | 79.6 | 7.38 | 7.32 | .00( | 31.53( | 26.3 | 43.5 |
| 631.8 | 114.8 | 7.66 | 7.44 | .00( | 32.67( | 26.9 | 42.9 |
| 689.4 | 170.6 | 7.51 | 7.47 | .00( | 32.46( | 26.3 | 43.1 |
| 740.0 | 256.7 | 7.44 | 7.49 | .00( | 32.19( | 25.9 | 43.2 |

G R L W E A P - Federal Highway Adm.

APPLE FREEWAY H-PILE @ ABUTMENT 05/10/93




| | | |
|---|---|---|
| DELMAG D 30-13 | | |
| Efficiency | .720 | |
| Helmet | 2.15 | kips |
| H Cushion | 39690 | k/in |
| Q = .100 | .100 | in |
| J = .050 | .100 | s/ft |
| Pile Length | 75.00 | ft |
| P-Top Area | 24.60 | in2 |




APPLE FREEWAY - HP12x84 DRIVEN THROUGH EMBANKMENT MATERIAL

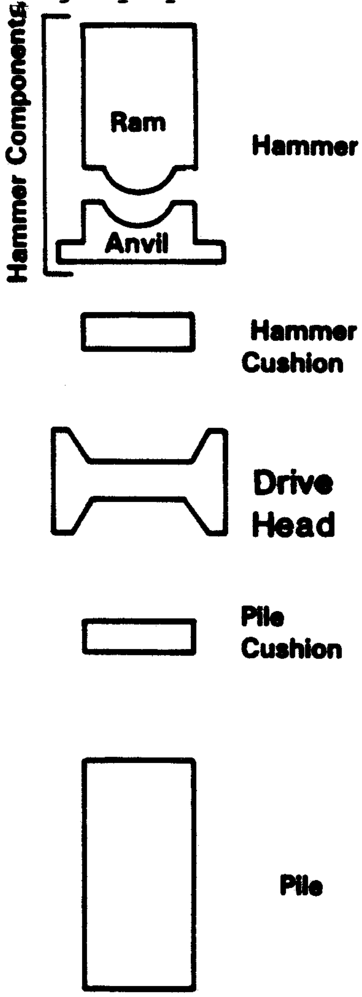
**SUMMARY OF DRIVEABILITY ANALYSES FOR HP12X84**

Pile section is adequate for the hardest driving conditions encountered at the east abutment. Driving stresses are below the maximum allowable driving stress of 32.4 ksi. Stresses are insensitive to the blow count and therefore, the pile won't be damaged when seated into the rock.

HP12X84 pile section is acceptable ✓

## ACCEPTANCE OF CONTRACTORS DRIVING SYSTEM

Contractor has submitted a ICE 70-S driving system. The pile and driving equipment data sheet is shown below.




**Hammer**

Manufacturer: ICE  Model: 70S
Type: OED  Serial No.: 123
Rated Energy: 70,000 lb-ft at 10 ft Length of Stroke
Modifications:

**Hammer Cushion**

Material: Micarta
Thickness 2 in  Area: 398 in²
Modulus of Elasticity – E 280,000 psi (P.S.I.)
Coefficient of Restitution-e 0.8

**Drive Head**

Helmet / Bonnet / Anvil Block / Pile Cap – Weight: 2.44 kips

**Pile Cushion**

Cushion Material: N/A
Thickness:  Area:
Modulus of Elasticity – E (P.S.I.)
Coefficient of Restitution

**Pile**

Pile Type: HP12X84
Length (In-Leads) – 75'
Weight/ft. 492 lb/ft³
Wall Thickness:  Taper: No
Cross Sectional Area 24.6 in² in²
* Design Pile Capacity: 120 (Tons)
Description of Splice:

Tip Treatment Description:

* Driving Resistance: 280.5 tons @ Pier
344.7 tons @ Embankments

Perform wave equation analysis for the submitted driving system.

- Modify hammer data in the GRLWEAP input file previously used to analyze the HP12×84 pile section. Use the submitted driving system data, and the GRLWEAP driveability option.

**GRLINP Input Screens for GRLWEAP Driveability option. Screens not Shown Below are Unchanged from the Pile Section Analysis.**

```
Title:  FULL EMBEDMT - DRIVABILITY OPTION                            Page:    1

ANALYSIS OPTIONS
     IOUT       IJJ      IHAMR      IOSTR      IFUEL       IPEL
    -100.        .        129.        .          .           .

ANALYSIS OPTIONS
        N       ISPL     NCROSS     IBEDAM     IPERCS     ISMITH     DMPEXP
        .         .         1.         .          .          .         .0

ANALYSIS OPTIONS
     ITYS       IPHI      IRSAO      ITER       IDAHA      IMAXT
        .         .         .          .          .          .

HELMET AND HAMMER CUSHION INFORMATION
   Helmet     ┌──────────────────── Hammer Cushion ─────────────────────────┐
   Weight      Area   ElasMod Thickness   C.O.R.  RoundOut  Stiffness
     2.44    398.00    280.0    2.000      .800     .0100        .0
```

**Continue GRLINP Input Screens for GRLWEAP Driveability Options Analysis of Ice 70s Driving System.**

```
Title:  FULL EMBEDMT - DRIVABILITY OPTION                            Page:    3

IPERCS = 0:  Friction Loss/Gain Factors
Give up to 10 Friction Loss/Gain Factors (5 on first line)
      .90      1.00      1.10       .00       .00

IPERCS = 0:  Friction Loss/Gain Factors
(5 on the second line)
      .00       .00       .00       .00       .00
```

Title: FULL EMBEDMT - DRIVABILITY OPTION Page: 6

IPERCS=0: DEPTHS & DRIVING SYSTEMS MODIFICATIONS ** PILE LENGTH: 75.00

| Analysis Depth | Stroke | IFUEL | Efficiency | Stiffn. Factor | Cushion CoR |
|---|---|---|---|---|---|
| 20.00 | .00 | .0 | .00 | .00 | .00 |
| 30.00 | .00 | .0 | .00 | .00 | .00 |
| 45.00 | .00 | .0 | .00 | .00 | .00 |
| 65.00 | .00 | .0 | .00 | .00 | .00 |
| 74.00 | .00 | .0 | .00 | .00 | .00 |
| 74.50 | .00 | .0 | .00 | .00 | .00 |
| 74.88 | .00 | .0 | .00 | .00 | .00 |
| 75.00 | .00 | .0 | .00 | .00 | .00 |
| .00 | .00 | .0 | .00 | .00 | .00 |

Title: FULL EMBEDMT - DRIVABILITY OPTION Page: 8

IPERCS = 0: SOIL PARAMETERS VS DEPTH ** PILE LENGTH: 75.00

| Depth | Skin Friction | End Bearing | Skin Quake | Toe Quake | Skin Damping | Toe Damping | Sens. |
|---|---|---|---|---|---|---|---|
| .00 | .000 | 5.000 | .100 | .100 | .050 | .150 | .000 |
| 30.00 | 8.830 | 58.000 | .100 | .100 | .050 | .150 | .000 |
| 30.00 | 2.200 | 5.000 | .100 | .100 | .200 | .150 | .000 |
| 65.00 | 2.200 | 5.000 | .100 | .100 | .200 | .150 | .000 |
| 65.00 | 12.080 | 96.000 | .100 | .100 | .050 | .150 | .000 |
| 75.00 | 13.720 | 351.000 | .100 | .100 | .050 | .150 | .000 |

## SUMMARY RESULTS OVER DEPTH FOR ICE 70S HAMMER

FRICTION LOSS/GAIN FACTOR: .900

| Depth (ft) | Rut (kips) | Frictn (kips) | End Bg (kips) | Bl Ct (bpf) | max Str (ksi) | min Str (ksi) | Bl Rte (b/min) | ENTHRU (kip-ft) |
|---|---|---|---|---|---|---|---|---|
| 20.0 | 83.7 | 48.4 | 35.3 | 4.1 | 15.493 | .000 | 45.4 | 34.8 |
| 30.0 | 170.1 | 114.4 | 55.7 | 8.5 | 25.056 | -.032 | 40.8 | 35.0 |
| 45.0 | 148.1 | 143.3 | 4.8 | 7.2 | 23.653 | -4.327 | 41.7 | 33.6 |
| 65.0 | 189.3 | 184.5 | 4.9 | 10.9 | 27.935 | -7.017 | 39.9 | 31.3 |
| *74.0* | *384.7* | *289.6* | *95.1* | *31.3* | *31.664* | *-.150* | *37.2* | *29.3* |
| *74.5* | *390.8* | *295.7* | *95.1* | *31.6* | *31.888* | *-.148* | *37.1* | *29.7* |
| *74.9* | *395.5* | *300.4* | *95.1* | *32.1* | *31.942* | *-.154* | *37.1* | *29.7* |
| *75.0* | *651.9* | *302.9* | *349.0* | *128.3* | *31.664* | *.000* | *36.8* | *30.0* |

Total Driving Time 15.72 min; Total No. of Blows 631

FRICTION LOSS/GAIN FACTOR: 1.000

| Depth (ft) | Rut (kips) | Frictn (kips) | End Bg (kips) | Bl Ct (bpf) | max Str (ksi) | min Str (ksi) | Bl Rte (b/min) | ENTHRU (kip-ft) |
|---|---|---|---|---|---|---|---|---|
| 20.0 | 89.6 | 54.1 | 35.5 | 4.3 | 15.098 | .000 | 45.3 | 34.0 |
| 30.0 | 183.3 | 127.5 | 55.8 | 9.7 | 24.582 | -.135 | 40.9 | 32.0 |
| 45.0 | 164.6 | 159.8 | 4.8 | 8.4 | 24.303 | -4.221 | 41.2 | 32.5 |
| 65.0 | 210.3 | 205.4 | 4.9 | 12.8 | 28.075 | -6.177 | 39.5 | 30.2 |
| ***74.0*** | ***417.1*** | ***322.0*** | ***95.1*** | ***35.1*** | ***32.602*** | ***-.115*** | ***37.0*** | ***29.5*** |
| ***74.5*** | ***423.9*** | ***328.8*** | ***95.2*** | ***36.1*** | ***32.693*** | ***-.069*** | ***37.0*** | ***29.4*** |
| ***74.9*** | ***429.1*** | ***334.0*** | ***95.2*** | ***36.8*** | ***32.740*** | ***-.052*** | ***37.0*** | ***29.4*** |
| ***75.0*** | ***685.8*** | ***336.7*** | ***349.1*** | ***158.6*** | ***32.103*** | ***.000*** | ***37.3*** | ***29.6*** |

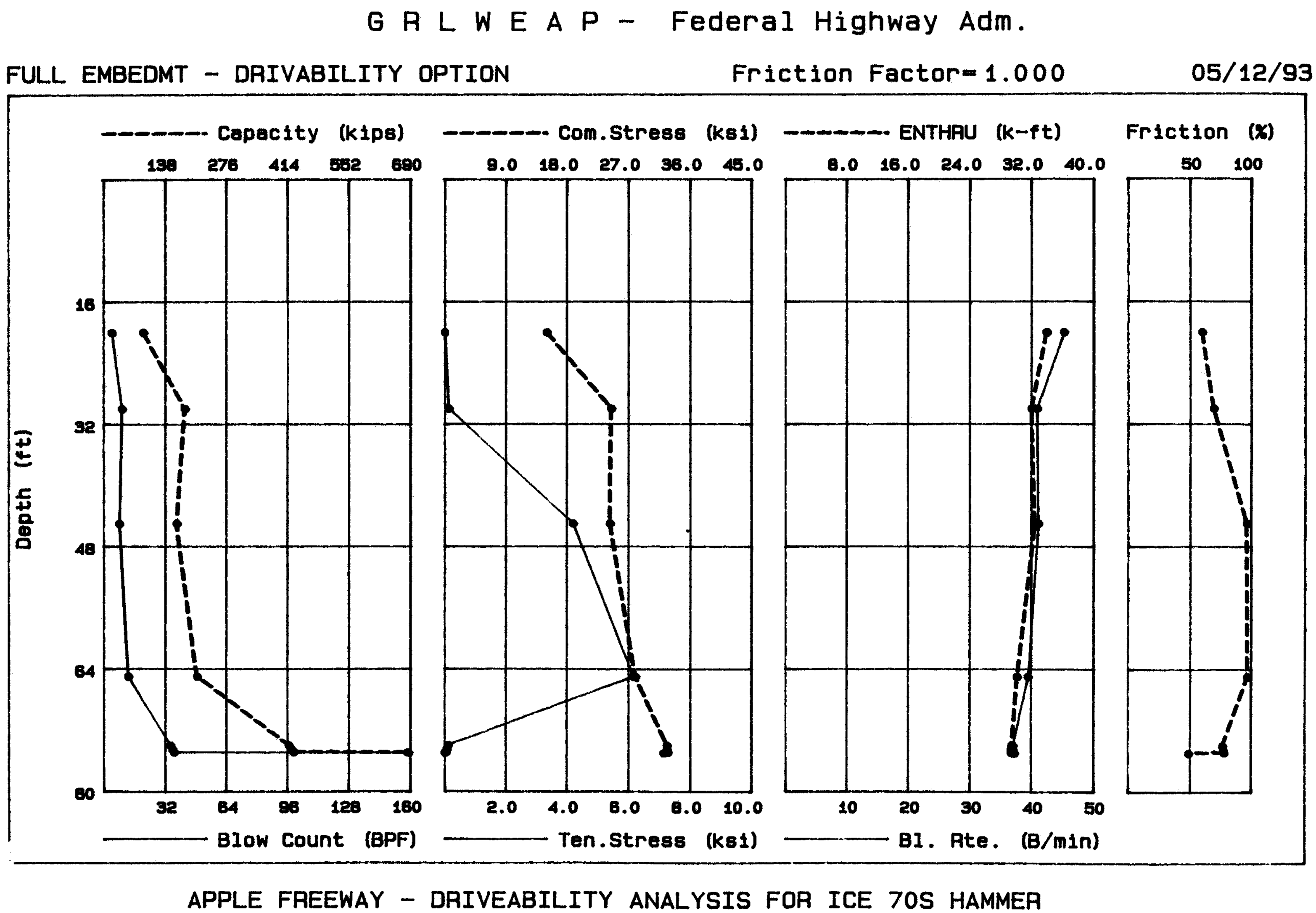
Total Driving Time 18.08 min; Total No. of Blows 720

FRICTION LOSS/GAIN FACTOR: 1.100

| Depth (ft) | Rut (kips) | Frictn (kips) | End Bg (kips) | Bl Ct (bpf) | max Str (ksi) | min Str (ksi) | Bl Rte (b/min) | ENTHRU (kip-ft) |
|---|---|---|---|---|---|---|---|---|
| 20.0 | 95.5 | 59.8 | 35.7 | 4.5 | 16.319 | .000 | 44.9 | 34.6 |
| 30.0 | 196.6 | 140.6 | 56.0 | 10.2 | 25.761 | -.978 | 40.6 | 32.9 |
| 45.0 | 181.2 | 176.3 | 4.8 | 9.3 | 26.005 | -4.863 | 40.7 | 32.7 |
| 65.0 | 231.2 | 226.3 | 4.9 | 14.3 | 29.119 | -5.837 | 39.0 | 30.6 |
| ***74.0*** | ***449.6*** | ***354.4*** | ***95.2*** | ***40.4*** | ***33.106*** | ***-.338*** | ***37.4*** | ***29.2*** |
| ***74.5*** | ***457.1*** | ***361.9*** | ***95.2*** | ***41.6*** | ***33.171*** | ***-.348*** | ***37.4*** | ***29.2*** |
| ***74.9*** | ***462.8*** | ***367.6*** | ***95.2*** | ***42.5*** | ***33.191*** | ***-.350*** | ***37.4*** | ***29.1*** |
| ***75.0*** | ***719.6*** | ***370.5*** | ***349.2*** | ***204.9*** | ***32.518*** | ***-.066*** | ***37.3*** | ***29.2*** |

Total Driving Time 20.19 min; Total No. of Blows 799

GRLWEAP Output – plot



**SUMMARY OF DRIVEABILITY ANALYSES FOR ICE 70S**

Driving Stresses: 32.7 ≈ 32.4 ksi ✓ OKAY

Driving stresses vary between 31.66 ksi (0.9 friction reduction) to 33.19 ksi (1.1 friction reduction), well below the yield strength of 36 ksi. Since, the maximum stresses occur when the pile has penetrated the rock and is at near refusal conditions, the piles should be capable of being seated into the rock without damage.

Blow Count: 159 bpf ≈ 144 bpf ✓ OKAY

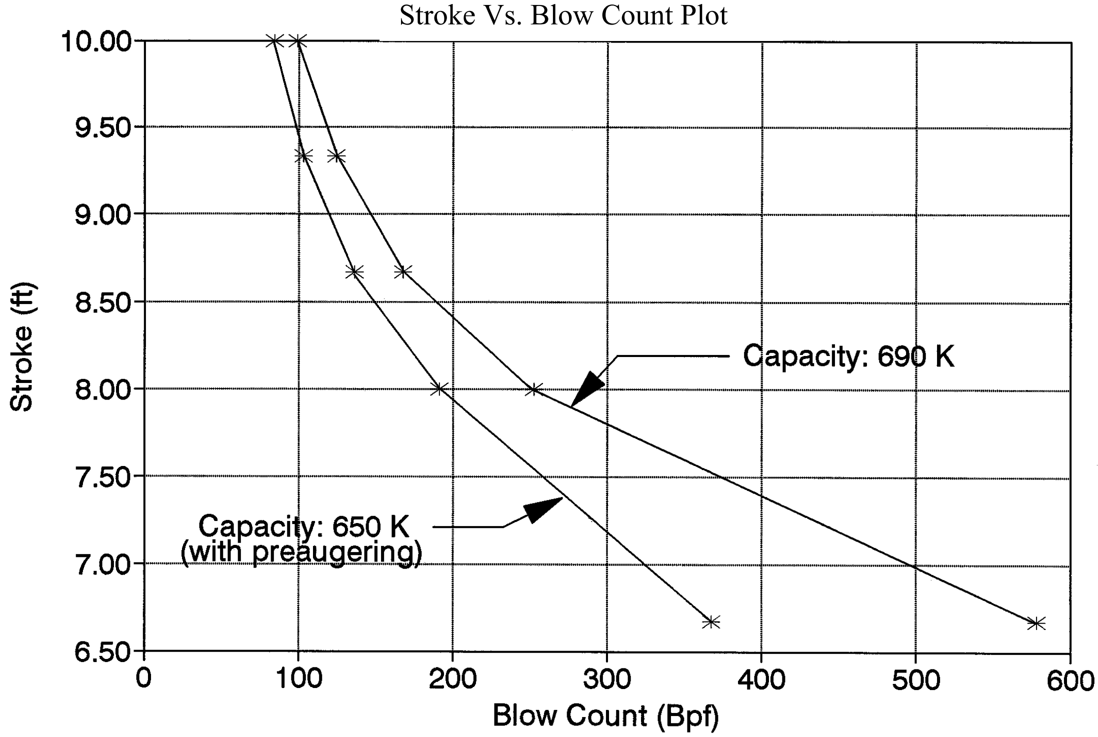
The blow count is approximately 35 bpf at just above the rock line, and near refusal 150 - 220 bpf in the rock layer. Therefore, the hammer should (if operating properly) penetrate quickly through the embankment and into the rock.

**HAMMER APPROVED ✓**

**PRODUCTION PILE DRIVING CRITERIA FOR ICE 70S DRIVING SYSTEM**

Drive HP12x84 pile through the embankment material and into the rock. Pile driving shall be terminated when the combination of stroke and blow count indicates a driving capacity of 690 kips. If preaugering is used the driving capacity of 650 kips should be attained.

Stroke Vs. Blow Count Plot



## CONSTRUCTION CONTROL

- Pile Driveability

  Driveability of 12 x 84 H-Pile

  Section verified for most difficult driving condition.

- Driveability versus Depth

  Driveability of 12 x 84 computed for full 75' depth.

  Pile installation time expected to vary between 16 and 20 minutes (no preaugering).

# CHAPTER 10.0
# FOUNDATION INVESTIGATION REPORT

Throughout the foundation design and construction process good communication and interaction should exist between the foundation engineer, structural engineer, and construction engineer. For example, even before the exploration program can be planned, the structural engineer should provide the foundation engineer with the proposed span arrangement and approximate structural loads. This interaction must be continued throughout the design and construction process to insure that the foundation design developed is compatible with the proposed structure type and is cost-effective. The importance of this communication and interaction cannot be overemphasized. Many design and construction problems are prevented through good communication.

The Foundation Investigation Report is the tool used to "communicate" the site conditions and design and construction recommendations to the bridge, roadway design, and construction engineers. The data from subsurface investigations usually are referred to continuously and for many different purposes during the design period, construction period, and, frequently, after completion of the project (resolving claims). Therefore, the foundation report should be clear, concise, and accurate. It is an extremely important document whose preparation deserves special care and effort.

## 10.1 GUIDELINES FOR WRITING A GOOD REPORT

The following guidelines apply to writing a good foundation report:

1. The soils engineer responsible for the report preparation should have a broad enough background in engineering to have some knowledge of the foundation requirements and limitations for various types of structures.

2. The report should contain an interpretation of subsurface conditions.

3. The report should contain specific engineering recommendations for design.

4. The materials and conditions which may be encountered during construction should be discussed.

5. The soils engineer should attempt to anticipate possible design and construction problems and make recommendations for their solution.

6. Recommendations given should be brief, concise, and, where possible, definite; don't be "wishy-washy."

7. Reasons and supporting data for recommendations should be included.

8. Extraneous data which are no use to the designer or project engineer should be omitted.

9. The report should include any special notes which should be placed on the plans or in the special provisions.

## 10.2 FOUNDATION REPORT OUTLINE

The following outline may be used as a general guide for presenting data in the foundation investigation report. The outline includes key items for which specific recommendations should be made, if pertinent to the given project.

A. Text

1. Introduction

2. Scope of Investigations
    a. Field explorations
    b. Lab testing

3. Interpretation of subsurface conditions.

4. Approach Embankment Considerations (Primarily for fills over soft weak subsoils).
    a. Stability
        1. Excavation and replacement of unsuitable materials
        2. Counterberm
        3. Stage construction - time delay
        4. Other treatment methods - change alignment, lower grade, lightweight fill, etc.
        5. Estimated factors of safety with and without treatment - estimated costs for treatment alternates - recommended treatment
    b. Settlement of subsoils
        1. Estimated settlement amount
        2. Estimated settlement time
        3. Surcharge height
        4. Special foundation treatment - vertical drains, soil densification, etc.
        5. Waiting periods
        6. Downdrag on piles
        7. Lateral squeeze of soft subsoils
    c. Construction considerations
        1. Select fill material - gradation and compaction requirements
        2. Construction monitoring (instrumentation)
    d. Special notes

5. Spread footing support
    a. Elevation of bottom of footing - based on frost depth, scour depth, or depth to competent bearing material
    b. Allowable soil pressure - based on settlement or bearing capacity: considering soil, adjacent foundations, water table, etc.
    c. Width of footing used in computations
    d. Special notes

6. Pile support
    a. Friction or end bearing or both
    b. Suitable pile types - reasons for choice and/or exclusion of types

c. Pile tip elevations
    1. Estimated tip elevation
    2. Specified tip elevation - explain reasons, such as underlying soft layers, negative skin friction, scour, piles uneconomically long, etc.
d. Estimated pile lengths
e. Allowable pile loads (design load)
f. Estimate of pile group settlement - only of practical significance for pile groups in cohesive soils and large groups in a cohesionless soil deposit underlain by compressible soils
g. Test piles to establish order lengths - specify test locations for maximum utility
h. Static pile load tests
i. Dynamic pile load tests (pile analyzer)
j. Driving criteria - based on dynamic pile formula or wave equation analysis
k. Corrosion effects - of particular concern in marine environments
1. Special notes
m. Actual driving resistance to reach estimated pile length

7. Drilled shaft support
    a. Shaft diameter
    b. Shaft length
    c. Allowable load
    d. Estimated settlement
    e. Load tests or integrity tests
    f. Special notes

8. Special design considerations
    a. Pile or drilled shaft lateral load capacity
    b. Seismic design - design earthquake ground acceleration, liquefaction potential (loose saturated sands and silts)
    c. Lateral earth pressures against retaining walls and high bridge abutments

9. Construction considerations
    a. Water table - fluctuations, control in excavation, pumping, tremie seals, etc.
    c. Excavations - safe slopes for open excavations, need for sheeting, shoring, etc.
    d. Drilled shafts - water table location, artesian water, boulders or obstructions, likely construction method (dry, casing, or slurry)
    e. Adjacent structures - protection against damage from excavation, pile driving, drainage, etc.
    f. Special notes.

B. Graphic Presentations

1. Map showing project location

2. Detailed plan of the site showing proposed structure(s), borehole locations, and existing structures

3. Laboratory test data

4. Finished boring logs and/or interpreted soil profile

Report Distribution

Copies of the completed Foundation Investigation Report should go to:

1. Bridge design section
2. Roadway design section
3. Construction section
4. Project engineer
5. Residency or maintenance group
6. Others as required by agency policy

## 10.3 TYPICAL SPECIAL CONTRACT NOTES

The foundation engineer should include in the Foundation Investigation Report any special notes which should be placed in the contract plans or special provisions. The purpose of such special notes is to bring the contractor's and/or project engineer's attention to certain requirements of the design or construction. Typical special notes relating to pile driving and embankment construction are as follows:

1. "Difficult driving of piles may be encountered and mechanical equipment may be necessary to remove consolidated material or boulders from the location of piles. This may be accomplished by various types of earth augers, well drilling equipment, or other devices to remove the consolidated material to permit piles to be driven to the desired depth or rated resistance without damage."

2. "If any obstructions to pile driving are encountered ten (10) feet or less from the bottom of the footing, the contractor shall, if so ordered by the engineer, pull the partially driven pile or piles and remove the obstruction, backfilling the hole with approved suitable material which shall be thoroughly compacted to the satisfaction of the engineer. However, no partially driven pile shall be removed until the engineer is satisfied that the contractor has made every effort to drive the pile through the obstruction. Payment for excavation will be made at the unit price bid for the Structure Excavation Item and for the temporary sheeting under Item _____ when sheeting is used. No other extra payment will be made for this work."

3. "The ordered length of pile shall be measured below the cut-off elevation shown on the plans. Any additional lengths of pile or splices above the cut-off elevation necessary to facilitate the contractor's operation shall be at his own expense."

4. "Piles for ______ are driven because of possible future scour of stream bed and shall be driven to the minimum lengths shown on the plans regardless of the resistance to driving. The actual driving resistance is estimated to be ___ tons."

5. "Piles will be acceptable only when driven to pile driving criteria established by the Chief Bridge Engineer. Prerequisite to establishing these criteria, the contractor shall submit, to the Chief Bridge Engineer, and others as required, Form _______ 'Pile and Driving Equipment Data'. All information

listed on Form ________ shall be provided within fourteen (14) days after the award of the contract. Each separate combination of pile and pile driving equipment proposed by the contractor will require the submission of a corresponding Form _______."

6. "Piles for the existing structure should be removed where they interfere with the pile driving for the new structure."

7. "It shall be the contractor's responsibility to place the cofferdams for ________ so that they will not interfere with the driving of batter piles. Pay lines for the cofferdams shall be as shown on the plans."

8. "The general subsurface conditions at the site of this structure are as shown on Drawing No._____."

9. "Pile driving will not be allowed at the abutments until fill settlement is complete. Estimated settlement time is _____ months after placement of the _____ foot surcharge."

10. "The contractor shall coordinate the project construction schedule to allow installation of embankment monitoring instrumentation by the State forces."

11. "Instrumentation damaged by contractor personnel shall be repaired or replaced at the contractor's expense. All construction activity in the area of any damaged instrument shall cease until the damage has been corrected."

12. "The contractor's attention is directed to the soil sample gradation test results which are shown on Drawing No._____. Soil sample gradation test results have been furnished to assist the contractor in determining dewatering procedures if necessary."

13. "The actual soil resistance to be overcome to reach estimated pile tip elevation is as shown below for each abutment and pier. The contractor shall size his pile driving equipment to install piles to the estimated length without damage."

14. "The south embankment shall be constructed to final grade and a month waiting period observed before pile driving begins. The actual length of the waiting period may be reduced by the Engineer based on an analysis of settlement platform and piezometer data."

## 10.4 SUBSURFACE INFORMATION MADE AVAILABLE TO BIDDERS

The finished boring logs and/or generalized soil profile should be made available to bidders and included with the contract plans. Other subsurface information, such as soil and rock samples and results of field and lab testing, should also be made available for inspection by bidders. The invitation for bids should indicate the type of information available and when and where it may be inspected. The highway agency should have a system for documenting what information each contractor inspects. Such documentation can be of major importance in later claim action.

The information developed during the foundation investigation is very useful in the selection of effective construction procedures, and for estimating construction costs. Such information is, therefore, of value to knowledgeable contractors bidding on the project. There has been much disagreement among owners and engineers as to what information should be made available to bidders, and how. The legal aspects are conflicting. In general, the owner's best interests are served by releasing pertinent information prior to the bid. Indeed, some courts have held that failure to reveal information can weaken the owner's position in the event of dispute. On the other hand, some engineers are fearful that the release of information will imply guarantees on their part that the information is fully representative of the actual conditions which will be encountered.

One of the best surveys of the problem has been prepared by Standing Subcommittee No. 4 of the U.S. National Committee on Tunneling Technology. The Subcommittee was composed of engineers and attorneys having experience dealing with owners, engineering firms, and contracting organizations.

The following is excerpted from their recommendations:

> "In sum, all subsurface data obtained for a project, professional interpretations thereof, and the design considerations based on these data and interpretations should be included in the bidding documents or otherwise made readily available to prospective contractors. Fact and opinion should be clearly separated.
>
> The bidder should be entitled to rely on the basic subsurface data, with no obligation to conduct his own subsurface survey.
>
> It is considered, however, that specific disclaimers of responsibility for accuracy are appropriate, with respect to the following categories:
>
> - Information obtained by others, perhaps at other times and for other purposes, which is being furnished prospective bidders in order to comply with the legal obligation to make full disclosure of all available data.
>
> - Interpretations and opinions drawn from basic subsurface data, because equally competent professionals may reasonably draw different interpretations from the same basic data."

Additional information on this topic is included in the FHWA Geotechnical Engineering Notebook; Geotechnical Guideline No. 15 – Geotechnical Differing Site Conditions.

## 10.5 USE OF DISCLAIMERS

The validity which courts give disclaimer clauses varies from State to State. In general, however, the courts have given much more validity to "specific" versus "general" disclaimer clauses. "General" disclaimer clauses are the type that say, in effect - subsurface information was gathered for use in design, however, the contractor should not rely on this information in preparing his bid. It is no big surprise, therefore, that judges give little validity to such general disclaimer clauses - since common sense dictates that if the subsurface information is good enough to base the design on, then the contractor should be able to place some reliance on the information in preparing his bid. Dr. Ralph Peck, noted geotechnical engineer, put it succinctly when asked his opinion concerning general disclaimer of subsurface information on a recent large Interstate project. He stated, "If the State or engineers it has engaged to develop the contract documents have accepted certain information as the basis for those documents, that information should not be disclaimed."

As mentioned previously, the courts have upheld the use of "specific" disclaimer clauses. The use of specific disclaimer clauses is strongly recommended over the use of a general disclaimer clause. An example of a specific disclaimer would be a statement such as - the boring logs are representative of the conditions at the location where the boring was made but conditions may vary between borings.

The following are examples of good "specific" disclaimer clauses used by one highway agency. These disclaimer clauses are placed on the interpreted soil profile which is included in the contract plans:

General Notes

1. The subsurface explorations shown hereon made between ______ and ______ by the regional soils section.

2. General soil and rock (where encountered) strata descriptions and indicated boundaries are based on an engineering interpretation of all available subsurface information by the Soil Mechanics Bureau and may not necessarily reflect the actual variation in subsurface conditions between borings and samples. Detailed data and field interpretation of conditions encountered in individual borings are shown on the subsurface exploration logs.

3. The observed water levels and/or conditions indicated on the subsurface profiles are as recorded at the time of exploration. These water levels and/or conditions may vary considerably, with time, according to the prevailing climate, rainfall or other factors and are otherwise dependent on the duration of and methods used in the explorations program.

4. Sound engineering judgment was exercised in preparing the subsurface information presented hereon. This information was prepared and is intended for State design and estimate purposes. Its presentation on the plans or elsewhere is for the purpose of providing intended users with access to the same information available to the State. This subsurface information interpretation is presented in good faith and is not intended as a substitute for personal investigation, independent interpretations or judgment of the contractor.

5. All structure details shown hereon are for illustrative purposes only and may not be indicative of the final design conditions shown in the contract plans.

6. Footing elevations shown are as indicated at the time of this drawing's preparation.

## 10.6 APPLE FREEWAY DESIGN EXAMPLE – FOUNDATION INVESTIGATION REPORT

A typical example of a Foundation Investigation Report is presented in the following section with reference to the Apple Freeway Design Example. The report illustrates the inclusion of various items discussed in the preceding sections of this chapter and summarize the pertinent results and conclusions obtained from the various analysis/design stages in the preceding chapters.

WORKSHOP DESIGN PROBLEM
FOUNDATION REPORT
July 4, 1993

Foundation Investigation Report

To: Mr. A. J. Jones
Chief Engineer

From: Mr. A. B. Smith
Chief Foundation Engineer

Subject: Interstate 0 Structure over the Apple Freeway

The Geotechnical section has completed an analysis of the foundation conditions at the site of the subject structure. Our analysis is based on the following information:

1. A 1-inch equals 20 feet plan and profile prepared by the Bridge Division and received in this office April 1, 1992.

2. An interpretation of the boring logs and analysis of soil samples from three drill holes numbered BAF-1 thru 3, nine auger holes numbered EA-1 thru 9, and one drill hole numbered BAF-4 from which undisturbed samples were taken.

3. Laboratory testing on undisturbed samples from BAF-4.

Subsurface Conditions:

The general subsurface conditions are shown on Drawing No. 5 GS 331.

Foundation Recommendations:

1. Elevation Assumptions

The foundation recommendations are based on the following bottom of footing elevations:

| | |
|---|---|
| West Abutment | 1011 |
| Pier | 992 |
| East Abutment | 1012 |

Changes to footing elevations may affect the foundation recommendations and should be discussed with this office.

2. Embankment Construction

A. Unsuitable Subexcavation

An approximate 1 to 3-foot thick organic layer exists between approximate stations 92+70 to 94+00 in the area of the east approach embankment. This organic layer should be removed and replaced with granular embankment material in accordance with Bridge Design Data Sheet 80-1.

B. Embankment Material and Placement

The approach embankment shall be constructed of materials placed in accordance with Bridge Design Data Sheet 80-1.

C. Embankment Settlement

An estimated 12 inches of fill settlement will occur due to consolidation of the 35-foot thick clay layer underlying the proposed 30-foot high east approach embankment. Estimated settlement time for 90 percent primary settlement is 14 months. Settlement time can be reduced to, (1) 6 months by use of a 10-foot surcharge fill or (2) 2 months through use of either 12-inch diameter sand drains at 9 foot center to center spacing or wick drains at 7.5 foot center to center spacing. Estimated cost for each of these treatments is:

| **Treatment** | **Estimated Settlement Time** | **Estimated Extra Cost** |
|---|---|---|
| Fill only | 14 months | $ --- |
| Fill w/10 foot surcharge | 6 months | 120, 000 |
| Fill w/wick drains | 2 months | 172,000 |
| Fill w/sand drains | 2 months | 385,000 |

It is understood the construction schedule will not allow a 14-month waiting period but will allow up to an 8-month waiting period, therefore, the 10-foot surcharge treatment is recommended as the most cost-effective method to reduce settlement time. The surcharge should be placed full height for a length of 500 feet back of the bridge ends on both the east and west approach and sloped at 1 vertical to 1.5 horizontal down to the embankment grade.

D. Embankment Stability

The estimated immediate end of construction factor of safety for the proposed 30' high east approach embankment is 1.63. The estimated immediate end-of-construction factor of safety for the proposed 30-foot fill plus 10 foot temporary surcharge is 1.33. Both factors of safety are adequate and no special approach embankment treatment is necessary. Long-term factor of safety will increase as consolidation of the foundation soils occur. The factor of safety for the west approach embankment will be higher as the fill height is 10' less. An analysis of highway borings confirms that no stability problems will occur due to the 500’ extension of the surcharge.

E. Embankment Monitoring

Fill settlement is recommended to be monitored with settlement plates and piezometers. Settlement plates should be installed at existing ground elevation at centerline stations 90+00, 93 + 50, and 96 + 50. Piezometers to monitor excess pore pressure buildup and dissipation in the clay subsoil are recommended at centerline stations 93 + 50 and 96 + 50. A total of three piezometers should be installed at each location - one each at 20, 28, and 36 foot depths. Instrumentation will be installed by State forces.

3. Abutment Foundation

A. Spread Footings

The abutments may be supported on spread footings placed on compacted select material with a maximum allowable bearing capacity of 3 tons per square foot assuming a footing width of 7' is used. Changes to the footing width affect both bearing capacity and settlement and should be discussed with this office. The total settlement of the east and west abutments respectively will be 2.6 and 1.9 inches which occurs over respective time periods of 14 months and 7 months. About 60 percent of the settlement will occur in 2 months after structure construction. This settlement may be reduced by extending the surcharge period. For 90 percent consolidation the surcharge should remain in place a total of 8 months; 2 months longer than required for embankment considerations. If vertical drains are installed during embankment construction, these drains will reduce the time for abutment settlement such that only ¼" will remain 30 days after all abutment loads have been placed

B. Piles

Two pile types were analyzed at the abutment; a displacement pile (12" diameter closed end pipe) and a non-displacement pile (12 x 84 H-pile). Displacement type piles are not recommended due to their inability to be driven through the fill and the uncertainty of obtaining penetration in the dense gravel stratum. Non-displacement H-piles are recommended. However, to insure that the pile can be driven to rock without damage, the section should not be less than a 12 x 84 H-pile. A 12 x 84 H-pile driven to rock may be designed for a maximum load of 120 tons. Tip reinforcement, such as APF 75500, should be used to prevent tip damage by boulders in the gravel stratum and to insure penetration to rock. Estimated pile lengths are 60 feet at the west abutment and 75 feet at the east abutment.

At the abutments, negative skin friction may be expected if the piles are installed before fill settlement is complete. In addition lateral squeeze of the clay subsoil will occur as the clay consolidates. Therefore, to prevent increased vertical downdrag pile loads, bending of abutment piles and rotation of the abutment toward the fill, the abutment piling should not be installed until embankment settlement is complete.

The actual driving resistance estimated to develop the design load for the H-pile at the estimated length is 345 tons at the east abutment and 290 tons at the west abutment. The contractor should size his equipment to achieve this resistance without damaging the pile.

4. Pier Foundation

A. Spread Footings

The pier may be supported on spread footings placed 4 feet below ground on natural undisturbed soil and designed for a maximum allowable bearing capacity of 3 tons per square foot assuming a footing width of 7' is used. Changes in footing width should be discussed with this office. Approximately 2.8 inches of settlement is expected at this location over about 7 months with 1 inch occurring immediately and 2 inches occurring in less than 2 months. If a spread footing foundation is chosen, consideration should be given to increasing the structure clearance over the Apple Freeway to account for these settlements. Settlement along the footing axes will be uniform. However a short term differential settlement of 1.5" can be expected between the abutment and pier footings if spread footings are used.

B. Piles

A 12" diameter closed end pipe pile and a 12 x 84 H-pile were analyzed at the pier. The closed end pipe may be designed for 70 tons with a safety factor of 2 if driven into the dense gravel layer. The estimated length is 36 feet. However a minimum wall thickness of 0.375 inches should be used to prevent overstress during driving. A driving resistance of 170 tons is estimated to reach the estimated length. A conical reinforced point should be used to prevent tip damage due to boulders. The cost per ton on a per foot basis equals $11.

A 12 x 84 H-pile may be designed for 120 tons with a safety factor of 2 if driven to rock. The estimated length is 46 feet. A driving resistance of 280 tons is estimated to obtain design resistance at the estimated length. A reinforced tip similar to APF 75500 should be used to prevent tip damage due to boulders. The cost per ton on a per foot basis equals $8.

We recommend that H-piles be chosen if piles are used because of cost advantages and installation advantages.

5. Special Notes

The following special notes are recommended to be included in the contract documents.

1. The general subsurface conditions at this site are shown on Drawing No. 5 GS 331.

2. A 6-month waiting period will be imposed between completion of the 10-foot surcharge on the east embankment. The actual length of the waiting period may be reduced by the Engineer based on an analysis of settlement platform and piezometer readings.

3. The contractor shall coordinate his construction schedule to allow installation of instrumentation by State forces.

4. Instrumentation damaged by contractor personnel shall be repaired or replaced at the contractor's expense. All construction activity in the area of any damaged instrument shall cease until the damage has been corrected.

If piles are used the additional special notes should be provided.

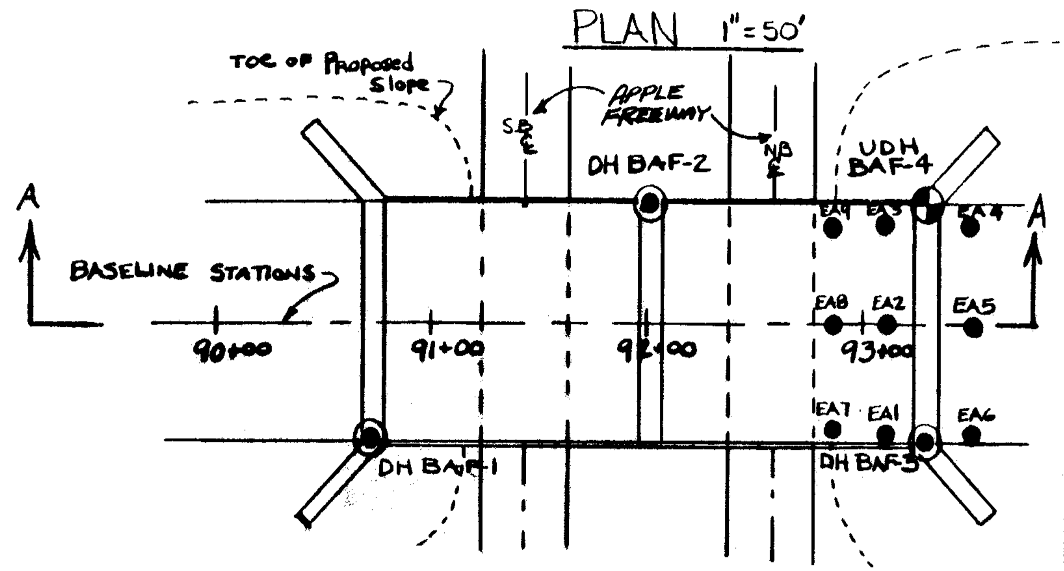
5. Pile driving will not be allowed at the abutments until fill settlement is complete. Estimated maximum settlement time is 6 months after placement of the 10-foot surcharge. This time may be reduced based on interpretation by the State of settlement plate readings.

6. Piles will be acceptable only when driven to pile driving criteria established by the Deputy Chief Engineer (Structures). Prerequisite to establishing these criteria, the contractor shall submit, to the Deputy Chief Engineer (Structures) and others as required, Form entitled, "Pile and Driving Equipment Data." All information listed on the Form shall be provided within 14 days after the award of the contract. Each separate combination of pile and pile driving equipment proposed by the contractor will require the submission of a corresponding Form.

7. The actual driving resistance to install the 12 x 84 H-piles to the estimated lengths shown on the plans is estimated to be 280 tons at the pier, 345 tons at the east abutment and 290 tons at

the west abutment. The contractor's equipment shall be capable of overcoming these resistances without inflicting pile damage.

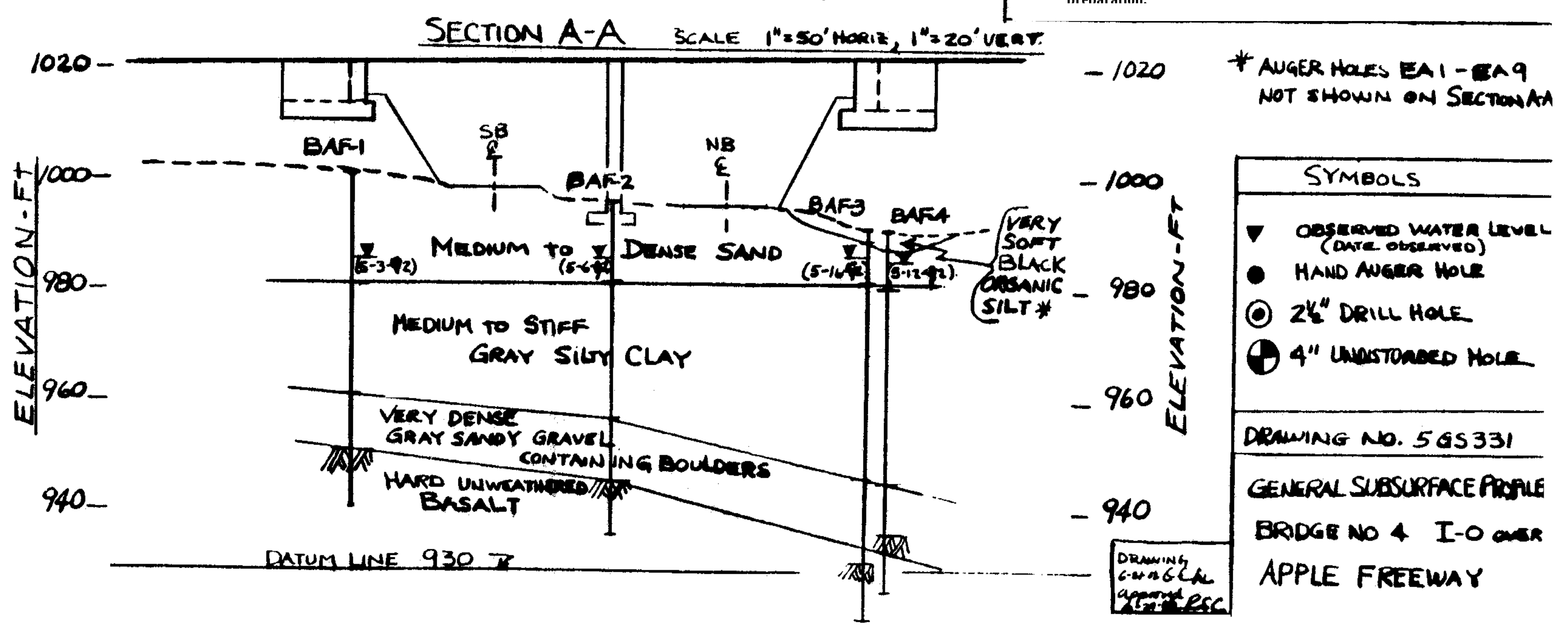
A B.Smith
Chief, Foundation Engineer




**General Notes**

The subsurface explorations shown hereon made between 5/1 and 5/12/92 by the regional soils section.

1. General soil and rock (where encountered) strata descriptions and indicated boundaries are based on an engineering interpretation of all available subsurface information by the Soil Mechanics Bureau and may not necessarily reflect the actual variation in subsurface conditions between borings and samples. Detailed data and field interpretation of conditions encountered in individual borings are shown on the subsurface exploration logs.
2. The observed water levels and/or conditions indicated on the subsurface profiles are as recorded at the time of exploration. These water levels and/or conditions may vary considerably, with time, according to the prevailing climate, rainfall or other factors and are otherwise dependent on the duration of and methods used in the explorations program.
3. Sound engineering judgment was exercised in preparing the subsurface information presented hereon. This information was prepared and is intended for State design and estimate purposes. Its presentation on the plans or else where is for the purpose of providing intended users with access to the same information available to the State. This subsurface information interpretation is presented in good faith and is not intended as a substitute for personal investigation, independent interpretations or judgment of the contractor.
4. All structure details shown hereon are for illustrative purposes only and may not be indicative of the final design conditions shown in the contract plans.
5. Footing elevations shown are as indicated at the time of this drawing's preparation.



# CHAPTER 11

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# APPENDIX A

**Modified Unified Description of Soil Samples ***

## Introduction

For many years the Unified Soil Classification System has been used successfully by soils engineers to categorize soil samples. The major advantage of this system is the easily understood word picture used to describe the soil samples after classification. The major disadvantage is the number of time consuming classification tests that must be done to develop the word picture.

At present, numerous private firms and State agencies are using the nomenclature of the Unified System but without the classification testing. This process of visually identifying and describing soil samples is known as the Modified Unified Description (MUD).

The procedure involves visually and manually examining soil samples with respect to texture, plasticity and color. A method is presented for preparing a "word picture" of a sample for entering on a subsurface exploration log or other appropriate data sheet. The procedure applies to soil descriptions made in the field or laboratory.

<u>It should be understood that the soil descriptions are based upon the judgement of the individual making the description. Classification tests are not intended to be used to verify the description, but to provide further information for analysis of soil design problems or for possible use of the soil as a construction material.</u>

It is the intent of this system to <u>describe only</u> the constituent soil sizes that have a significant influence on the visual <u>appearance and behavior</u> of the soil. This description system is intended to provide the best word description of the sample to those involved in the planning, design, construction, and maintenance processes.

| Terms | Definition of Terms |
|---|---|
| Boulder | A rock fragment, usually rounded by weathering or abrasion, with average dimension of 12 inches or more. |
| Cobble | A rock fragment, usually rounded or subrounded, with an average dimension between 3 to12 inches. |
| Gravel | Rounded, subrounded, or angular particles of rock that will pass a 3-inch square opening sieve (76.2 mm) and be retained on a Number 4 U.S. standard sieve (4.76 mm).<br><br>(The term "gravel" in this system denotes a particle size range and should not be confused with "gravel" used to describe a type of geologic deposit or a construction material.) |
| Sand | Particles that will pass the Number 4 U.S. standard sieve and be retained on the Number 200 U.S. standard sieve (0.074 mm). |
| Silt | Material passing the Number 200 U.S. standard sieve that is nonplastic and exhibits little or no strength when dried. |
| Clay | Material passing the Number 200 U.S. standard sieve that can be made to exhibit plasticity (putty like property) within a wide range of water contents and exhibits considerable dry strength. |
| Fines | The portion of a soil passing a Number 200 U.S. standard sieve. |
| Marl | Unconsolidated white or dark gray calcium carbonate deposit. |
| Muck | Finely divided organic material containing various amounts of mineral soil. |
| Peat | Organic material in various stages of decomposition. |
| Organic Clay | Clay containing microscopic size organic matter.  May contain shells and/or fibers. |
| Organic Silt | Silt containing microscopic size organic matter.  May contain shells and/or fibers. |
| Coarse Grained Soil | Soil having a predominance of gravel and/or sand. |
| Fine Grained Soil | Soil having a predominance of silt and/or clay. |
| Mixed-Grained Soil | Soil having significant proportions of both fine-grained and coarse-grained sizes. |

NOTE: When applied to gradation test results, silt size is defined as that portion of the soil finer than the No. 200 U.S. standard sieve and coarser than 0.002 mm.  Clay size is that portion of soil finer than 0.002 mm.  For the visual-manual procedure the identification will be based on plasticity characteristics.

| Visual – Manual Identification | |
|---|---|
| Gravel | Identify by particle size. The particles may have a angular, rounded, or subrounded shape. Gravel size particles usually occur in varying combinations with other particle sizes. |
| Sand | Identified by particle size. Gritty grains that can easily be seen and felt. No plasticity or cohesion. Size ranges between gravel and silt. |
| Silt | Identified by behavior. Fines that have no plasticity. May be rolled into a thread but will easily crumble. Has no cohesion. When dry, can be easily broken by hand into powdery form. |
| Clay | Identified by behavior. Fines that are plastic and cohesive when in a moist or wet state. Can be rolled into a thin thread that will not crumble. When dry, forms hard lumps which cannot be readily broken by hand.<br><br>Clay is often encountered in combination with other soil sizes. A sample which exhibits plasticity or cohesion contains clay. The amount of clay can be related to the degree of plasticity or cohesiveness; the higher the clay content the greater the plasticity. |
| Marl | A white or gray calcium carbonate paste. May contain granular spheres, shells, organic material or inorganic soils. Reacts with weak hydrochloric acid. |
| Muck | Black or dark brown finely divided organic material mixed with various proportions of sand, silt, and clay. May contain minor amounts of fibrous material such as roots, leaves, and sedges. |
| Peat | Black or dark brown plant remains. The visible plant remains range from coarse fibers to finely divided organic material. |
| Organic Clay | Dark gray clay with microscopic size organic material dispersed throughout. May contain shells and/or fibers. Has weak structure which exhibits little resistance to kneading. |
| Organic Silt | Dark gray silt with microscopic size organic material dispersed throughout. May contain shells and/or fibers. Has weak structure which exhibits little resistance to kneading. |
| Fill | Man-made deposits of natural soils and/or waste materials. Document the components carefully since presence and depth of fill are important engineering considerations. |

| Soil Sample Identification Procedure | |
|---|---|
| 1st Decision | Is sample coarse-grained, fine-grained, mixed-grained or organic?<br>If mixed-grained, decide whether coarse-grained or fine-grained predominates. |
| 2nd Decision | What is principal component?<br>Use a <u>noun</u> in soil description. Example: Sand. |
| 3rd Decision | What is secondary component?<br>Use as <u>adjective</u> in soil description. Example: Silty Sand. |
| 4th Decision | Are there additional components?<br>Use as additional adjectives. Example: Silty Sand, Gravelly. |

| Example of Description of the Soil Components | |
|---|---|
| Sand | Describes a sample that consists of <u>both</u> fine and coarse sand particles. |
| Gravel | Describes a sample that consists of <u>both</u> fine and coarse gravel particles. |
| Silty Fine Sand | Major component fine sand, with non-plastic fines. |
| Sandy Gravel | Major component gravel size, with fine and coarse sand. May contain small amount of fines. |
| Gravelly Sand | Major component sand, with gravel. May contain small amount of fines. |
| Gravelly Sand, Silty | Major component sand, with gravel and non-plastic fines. |
| Gravelly Sand, Clayey | Major component sand, with gravel and plastic fines. |
| Sandy Gravel, Silty | Major component gravel size, with sand and non-plastic fines. |
| Sandy Gravel, Clayey | Major component gravel size, with sand and plastic fines. |
| Silty Gravel | Major component gravel size, with non-plastic fines. May contain sand. |
| Clayey Gravel | Major component gravel size, with plastic fines. May contain sand and silt. |
| Clayey Silt | Major component silt size, with sufficient clay to impart plasticity and considerable strength when dry. |
| Silty Clay | Major component clay, with silt size. Higher degree of plasticity and higher dry strength than clayey silt. |

The above system may be expanded where necessary to provide meaningful descriptions of the sample.
Examples: Shale fragments - Cobble and gravel size, silty
Decomposed rock - Gravel size

| Other Information for Describing Soils | |
|---|---|
| Color of the Sample | Brown, Gray, Red, Black, etc. |
| Moisture Condition | Dry, Moist, Wet. Judge by appearance of sample before manipulating. |
| Plasticity | Plastic, Low Plastic, Non-plastic. Sample must be in moist or wet condition for plasticity determination. For dry samples requiring wetting make note in description. Example - "plastic (low or non-plastic) when wet." Plasticity not required for marl, muck and peat. |
| Structure | Fissured, Blocky, Varved, Layered. (Indicate approximate thickness of layers). The description of layering for coarse-grained soils must be made from field observations before sample is removed from sampler. |
| Particular Shape | Angular, Rounded, Subrounded. |
| Other words, phrases, notes or remarks that will add to the meaningfulness of the complete soil description. | |

## Preparing the Word Picture

The word-picture is the description of the soil sample as determined by the visual-manual procedure. Where applicable, the following are to be included in the word-picture:

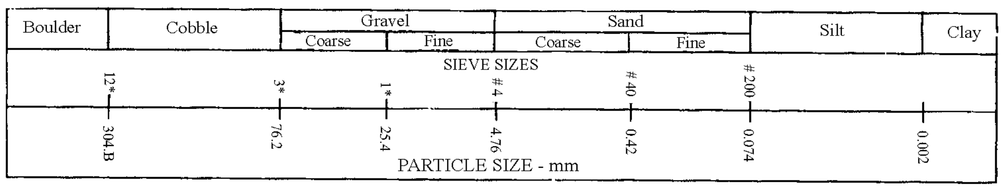
| Pertinent Information | Example |
|---|---|
| Color of the sample | Brown |
| Description of soil components | Silty gravel |
| Moisture condition | Moist |
| Plasticity | Non-plastic |
| Structure | Blocky |
| Particle Shape | Angular |
| Other | Cemented |

The written description for the given example is: Brown Silty Angular Gravel, Moist, Non-plastic, and Cemented.

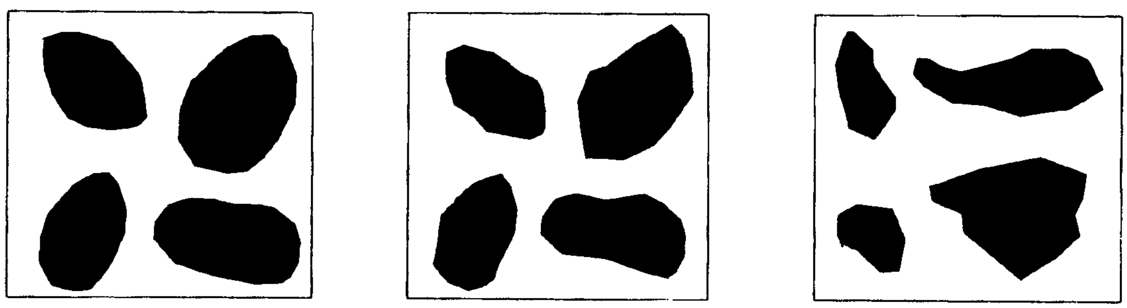
Examples of Complete Soil Descriptions

- Light Gray Silty Clay, moist, plastic, with ½ inch layers of wet gray silt, non-plastic.
- Red brown Clayey silt with ¼ inch layers of Silty clay, moist, plastic
- Brown Silty fine Sand, wet, non-plastic
- Gray Sandy rounded Gravel, dry non-plastic
- Gray Sandy angular Gravel, Clayey, moist, low plastic
- Dark Brown Silty Sand, wet, non-plastic
- Red Brown Silty Sand, wet, non-plastic

- Fill – Brown Sandy subrounded Gravel, with pieces of brick and cinders, wet, non-plastic.
- Fill containing cinders, paper, garbage, and glass, wet
- Dark Gray Organic Clay, with Shells and roots, moist, plastic.




PARTICLE SIZE LIMITS



Rounded Subrounded Angular

COARSE – GRAINED SOILS
(Identify by Size of Particles)

BOULDER
Greater Than 12″

COBBLES
12″ to3″

GRAVEL
3″ to #4
Coarse - 3″ to 1″
Fine - 1″ to #4

Sand
# 4 to # 200
Coarse - # 4 to # 40
Fine - # 40 to # 200

FINE - GRAINED SOILS
(Identify by Behavior –
Individual Particles Not Visible)

SILT
1. Nonplastic
2. Powders easily when dry
3. Dries rapidly
4. Free water appears when shaken
5. Wire cut surface – rough

CLAY
1. Plastic, acts like putty when moist and wet
2. High dry strength
3. Dries slowly – sticky
4. No free water when shaken
5. Wire cut surface – smooth

MIXED – GRAINED SOILS
Significant proportions of coarse – grained and fine – grained sizes. Make judgement on whether fine-grained or coarse – grained predominates in behavior

ORGANIC SOILS
(Identify by Appearance, Behavior, Color, Etc.)

Peat

Muck

Marl

Organic Silt

Organic Clay

# APPENDIX B

# Unified Soil Classification System

| Primary Divisions for Field and Laboratory Identification | | | Group Symbol | Typical Names | Laboratory Classification Criteria | | Supplementary Criteria for Visual Identification |
|---|---|---|---|---|---|---|---|
| …do… | …do… | Gravels with fines (More than 12% of material smaller than No. 200 sieve size)* | GM | Silty gravels, and gravel-sand-silt mixtures | Atterberg limits below "A" line, or PI less than 4 | Atterberg limits above "A" line with PI between 4 & 7 is borderline case GM-GC | Nonplastic fines or fines of low plasticity |
| | | | GC | Clayey gravels, and gravel-sand-clay mixtures | Atterberg limits above "A" line, and PI greater than 7 | | Plastic fines |
| …do… | Sands (More than half of the coarse fraction is smaller than No. 4 sieve size) | Clean sands (Less than 5% of material smaller than No. 200 sieve size) | SW | Well graded sands, gravelly sands, little or no fines* | $C_U = \frac{D_{60}}{D_{10}}$ greater than 6 $C_Z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 | | Wide range in grain sizes and substantial amounts of all intermediate particle sizes |
| | | | SP | Poorly graded sands and gravely sands, little or no fines* | Not meeting both criteria for SW | | Predominately one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded) |

*Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

# Unified Soil Classification System

<table>
<tr><th colspan="3">Primary Divisions for Field and Laboratory Identification</th><th>Group Symbol</th><th>Typical Names</th><th colspan="2">Laboratory Classification Criteria</th><th>Supplementary Criteria for Visual Identification</th></tr>
<tr><td rowspan="2">…do…</td><td rowspan="2">…do…</td><td rowspan="2">Sands with fines (More than 12% of material smaller than No. 200 sieve size.)*</td><td>SM</td><td>Silty sands, sand-silt mixtures</td><td>Atterberg limits below “A” line, or PI less than 4</td><td rowspan="2">Atterberg limits about “A” line with PI between 4 and 7 is border line case SM-SC</td><td>Nonplastic fines or fines of low plasticity</td></tr>
<tr><td>SM</td><td>Clayey sands, sand-clay mixures</td><td>Atterberg limits aboe “A” line with PI greater than 7</td><td>Plastic fines</td></tr>
</table>

***Materials with 5 to 12 percent smaller then No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.**

# Unified Soil Classification System

<table>
<tr><th colspan="2">Primary Divisions for Field and Laboratory Identification</th><th>Group Symbol</th><th>Typical Names</th><th colspan="2">Laboratory Classification Criteria</th><th colspan="3">Supplementary Criteria for Visual Identification</th></tr>
<tr><td rowspan="4">Fine-grained soils (More than half of material is smaller than No. 200 sieve size)(Visual: more than half of particles are so fine that they can not be seen by the naked eye)</td><td rowspan="2">Silts and clays (Liquid limit less than 50)</td><td rowspan="2">ML</td><td rowspan="2">Inorganic silts, very fine sands, rock flour, silty or clayey fine sands</td><td rowspan="2">Atterberg limits below “A” line, or PI less than 4</td><td rowspan="3">Atterberg limits above “A” line with PI between 4 and 7 is borderline case ML-CL</td><td>Dry strength</td><td>Reaction to shaking</td><td>Toughness near Plastic Limit</td></tr>
<tr><td>None to slight</td><td>Quick to slow</td><td>None</td></tr>
<tr><td>…do…</td><td>CL</td><td>Inorganic clays of low to medium plasticity; gravelly clays, silty clays, sandy clays, lean clays</td><td>Atterberg limits above “A” line with PI greater than 7</td><td>Medium to high</td><td>None to very slow</td><td>Medium</td></tr>
<tr><td>…do…</td><td>OL</td><td>Organic silts and organic silt-clays of low plasticity</td><td colspan="2">Atterberg limits below “A” line</td><td>Slight to medium</td><td>Slow</td><td>Slight</td></tr>
</table>

# Unified Soil Classification System

| Primary Divisions for Field and Laboratory Identification | | Group Symbol | Typical Names | Laboratory Classification Criteria | Supplementary Criteria for Visual Identification | | |
|---|---|---|---|---|---|---|---|
| | | | | | Dry Strength | Reaction to Shaking | Toughness Near Plastic Limit |
| …do… | Silts and clays (Liquid limit greater than 50) | MH | Inorganic silts, micaceous of diatomaceous fine sands or silt, elastic silts | Atterberg limits below “A” line | Slight to medium | Slow to none | Slight to medium |
| | …do… | CH | Inorganic clays of high plasticity, fat clays | Atterberg limits above “A” line | High to very high | None | High |
| | …do… | OH | Organic clays of medium plasticity | Atterberg limits below “A” line | Medium to high | None to very high | Slight to medium |
| …do… | Highly organic soils….. | Pt | Peat, muck and other highly organic soils | High ignition loss, LL and PI decrease after drying | Organic color and odor, spongy feel, frequently fibrous texture | | |

# Unified Soil Classification System

| Primary Divisions for Field and Laboratory Identification | | | Group Symbol | Typical Names | Laboratory Classification Criteria | Supplementary Criteria for Visual Identification |
|---|---|---|---|---|---|---|
| Coarse grained soils (More than half of material finer than 3-inch sieve is larger than No. 200 sieve size) | Gravel (More than half of the coarse fraction is larger than No. 4 sieve size about ¼ inch) | Clean gravels (Less than 5% of material smaller than No. 200 sieve size) | GW | Well graded gravels, gravel-sand mixtures, little of no fines* | $C_U = \frac{D_{60}}{D_{10}}$ greater than 4 $C_Z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 | Wide range in grain size and substantial amounts of all intermediate particle size |
| | | | GP | Poorly graded gravels, gravel-sand mixtures, little or no fines* | Not meeting both criteria for GW | Predominantly one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded) |

*Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

# APPENDIX C

## EMBANKMENT IN PLACE (LIGHTWEIGHT FILL)

### Description

Under this Item, the Contractor shall furnish and place lightweight fill necessary to complete the embankments shown on the plans or as ordered by the Engineer.

### Materials

The material shall be blast furnace slag, expanded shale or other materials as approved by the Deputy Chief Engineer, Technical Services. The material shall have a maximum particle size of 24 inches in greatest dimension and the compacted wet density shall not exceed the density specified in the proposal as measured in a test embankment.

### Construction Details

The compacted wet density shall be determined in test embankments containing a minimum of 400 cubic yards of material constructed on firm flat surfaces. The Contractor shall construct each test embankment in an area bounded by 100 ft. by 50-ft. dimensions and shall give the Engineer at least one (1) week written notice prior to beginning each test in order for the location to be inspected and surveyed.

The lightweight fill material shall be stored in piles not exceeding 20,000 cubic yards prior to testing. Representative material from each storage pile shall be used to construct a test embankment to a minimum height of four (4) feet in accordance with this specification.

The Contractor shall weigh all the material prior to placement in the test embankment. The embankment shall be constructed in uniform layers not exceeding 24 inches in thickness prior to compaction. Each layer shall be rolled over its entire area by a vibratory steel drum roller. The number of passes, the size of vibratory steel drum roller, and the need for actually vibrating the roller will be as directed by the Engineer.

The Engineer shall determine the volume of the test embankment. If the compacted wet density of the material in the test embankment is greater than the specified density, both the material contained in the test embankment and the material from the storage pile it represents shall be rejected for use under this Item.

The design embankment shall be constructed using the same methods, equipment and procedures used to construct the test embankments. However, the following requirements contained in the earthwork section shall now apply:

a. The density requirements both in the embankment and in the subgrade area.

b. The maximum particle size in the subgrade area.

c. Proof rolling.

d. Compaction.

The top surface of the lightweight embankment lying directly beneath the subbase course materials shall be chinked to the satisfaction of the Engineer with lightweight material to prevent infiltration of the subbase materials.

**Method of Measurement**

The quantity of lightweight fill to be paid for under this Item shall be the number of cubic yards of material computed in its final compacted position between the payment lines shown on the plans or between revised payment lines established by the Engineer prior to performing the work.

**Basis of Payment**

The unit price bid per cubic yard shall include the cost of furnishing all labor, material and equipment necessary to complete the work including the test embankments.

No payment will be made for any loss of material which may result from foundation settlement, erosion or any other cause.  The cost of such losses shall be included in the price bid for this item.

# APPENDIX D

# <u>Lightweight Fill - Sawdust</u>

The following is the special provision for lightweight sawdust fill used by the Washington State DOT.

**Sawdust borrow in place**

Where shown in the plans or where directed by the Engineer, the Contractor shall furnish, load, haul, place, and compact sawdust borrow in place.

**Materials**

The sawdust borrow shall consist of 100 percent wood fibers, such as sawdust, hog fuel or wood chips. No composition wood products, such as particle or chip board, pressed hard board, or presto-log fragments shall be used in this embankment. Maximum size shall be 6 inches in the greatest dimension. Sufficient smaller sized material shall be used to produce a uniformly dense fill. Cedar sawdust borrow will not be allowed.

**Construction**

The sawdust borrow embankments may be constructed by dumping from trucks or by any other methods approved by the Engineer. Sawdust borrow shall be placed in lifts a maximum of 1 foot in depth of uncompacted material.

Compaction shall be obtained by covering the entire surface of each lift with a minimum of two passes with a D8-Caterpillar tractor or other similar compaction units as approved by the Engineer. Hauling units shall be routed over the entire fill for additional compaction.

**Measurement**

Sawdust borrow in place will be measured by the cubic yard of neat line volume in place.

**Payment**

The unit contract price per cubic yard for "Sawdust Borrow in Place" shall be full compensation for furnishing all labor, tools, equipment and materials necessary or incidental to complete the work as specified, including loading, hauling, placing, and compacting.

# APPENDIX E

**Typical Specification for Select Material**

DESCRIPTION – This work shall consist of excavation, disposal, placement, and compaction of all materials that are not provided for under another section of these specifications, and shall be executed in conformance with payment lines, grades, thickness, and typical sections specified in the contract documents.

MATERIALS – Tests and Control Methods. Materials tests and control methods pertaining to the item requirements and work of this section will be performed in conformance with the procedures used by the Department.

Materials furnished under these items shall conform to the following requirements:

1. Gradation – The material shall have the following gradation:

| Sieve Size | Percent Passing by Weight |
|---|---|
| 4 inches | 100 |
| No. 40 | 0 – 70 |
| No. 200 | 0 – 15 |

2. Soundness – The material shall be substantially free of shale or other soft, poor durability particles. Where the State elects to test for this requirement, a material with a magnesium sulfate soundness loss exceeding 30 percent after four (4) cycles will be rejected.

CONSTRUCTION – The type of material to be used in filling and backfill at structures, and payment lines, therefore, shall be in conformance with the details shown on the appropriate Standard Sheet or as noted on the plans or as ordered by the engineer.

Fill or backfill material at structures, culverts, and pipes shall be deposited in horizontal layers not exceeding 6 inches in thickness prior to compaction. Compaction of each layer shall be as specified. A minimum of 100 percent of standard Proctor maximum density (AASHTO T99) will be required. When filling behind abutments and similar structures, all material shall be placed and compacted in front of the walls prior to placing fill behind the walls to a higher elevation. The limits to which this subsection will apply, shall be in accordance with the Standard Sheets or as modified on the plans.

MEASUREMENT – Quantities for this work shall be computed in cubic yards in the final compacted position. A deduction shall be made for pipes (based on nominal diameters) and other payment items when the combined cross-sectional area exceeds one square foot unless otherwise shown on plans. No deduction will be made for the cross-sectional area of an existing facility.

BASIS OF PAYMENT – The unit price bid for all pay items of work encompassed by this section, shall include the costs of furnishing all equipment, labor, and materials as necessary to complete the work of the item, except where specific costs are designated or included in another pay item of work. All incidental costs, such as acquisition of borrow pits or material outside of the right-of-way, rock drilling and blasting, compaction and special test requirements, stockpiling, and rehandling of materials, precautionary measures to protect private property and utilities and to form and trim graded surfaces, shall be included in the unit price of the pay item where such costs are incurred.

# APPENDIX F

**Typical Specifications for Underdrain Filter Material**

DESCRIPTION – The work shall consist of constructing underdrain filter installations in accordance with these specifications and in conformity with the plans.

MATERIAL – Underdrain Filter Material shall consist of crushed stone, sand, gravel, or screened gravel. Material tests and quality control methods pertaining to the item requirements and work of this Section will be performed in conformance with the procedures in use by the Department.

Underdrain Filter Material shall be stockpiled.
Gradation:

| Sieve Size | Percent Passing by Weight |
|---|---|
| 1 inch | 100 |
| ½ inch | 30 – 100 |
| ¼ inch | 0 – 30 |
| No. 10 | 0 – 10 |
| No. 20 | 0 – 5 |

Soundness: The soundness of the material shall be tested. This material shall have a loss not exceeding 20 percent by weight after four (4) cycles of the magnesium sulphate soundness test.

CONSTRUCTION DETAILS – Underdrain Filter material, shall be placed adjacent to structures as specified on the contract plans. The lift thickness for the loose material shall not exceed six (6) inches and shall precede the placement of each lift of adjacent backfill material. A physical barrier may be used to facilitate placement of the Underdrain Filter and adjacent backfill. This barrier shall not be left in place and shall be removed prior to compaction of the material. Each lift of filter material and backfill material located within a minimum distance of 3 feet from the back wall plus the footing heel projection shall be compacted simultaneously. Compactive effort for this material shall be provided by two passes of vibratory compactor approved by the engineer. Placement and compaction operations shall be conducted in a manner so as to insure that the top surface of each lift of filter material shall not be contaminated by the adjacent backfill materials. No compaction control tests will be required for the filter material.

METHOD OF MEASUREMENT – The quantity of Underdrain Filter material shall be computed for payment as the number of cubic yards placed between the payment lines shown on the contract Plans or as modified by the engineer. No deduction will be made for the volume occupied by the underdrain pipe.

BASIS OF PAYMENT – The unit price bid per cubic yard shall include the cost of furnishing all labor, materials, and equipment necessary to complete the work. No direct payment will be made for any losses of material which may result from compaction, foundation settlement, erosion, or any other causes; the cost of such losses shall be included in the price bid for this item. Any contaminated underdrain filter material shall be replaced by the contractor as directed by the engineer at no cost to the State. Excavation, granular fill, and backfill will be paid for separately under their appropriate items.

# APPENDIX G

**Example Specification for Bitumen Coating**

**Description**

This work shall consist of furnishing and applying bituminous coating and primer to prestressed concrete pile surfaces as required in the plans and as specified herein.

**Materials**

A. Bituminous Coating. Bituminous coating shall be an asphalt type bitumen conforming to ASTM D946, with a minimum penetration grade 50 at the time of pile driving. Bituminous coating shall be applied uniformly over an asphalt primer. Grade 40-50 or lower grades shall not be used.

B. Primer. Primer shall conform to the requirements of ASTM D41.

**Construction Requirements**

All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather, nor when the temperature is below 65 degrees F.

The primer shall be applied to the surfaces and allowed to completely dry before the bituminous coating is applied. Primer shall be applied uniformly at the quantity of one gallon per 100 square feet of surface.

Bitumen shall be applied uniformly at a temperature of not less than 300 degrees F., nor more than 350 degrees F. and shall be applied either by mopping, brushing, or spraying at the project site. All holes or depressions in the concrete surface shall be completely filled with bitumen. The bituminous coating shall be applied to a minimum dry thickness of 1/8 inch but in no case shall the quantity of application be less than 8 gallons per 100 square feet.

Bitumen coated piles shall be stored before driving and protected from sunlight and heat. Pile coatings shall not be exposed to damage during storage, hauling or handling. The Contractor shall take appropriate measures to preserve and maintain the bitumen coating. At the time of pile driving, the bitumen coating shall have a minimum dry thickness of 1/8 inch and a minimum penetration value of 50. If necessary, the Contractor shall re-coat the piles, at his expense, to comply with these requirements.

**Method of Measurement**

Bitumen coating will be measured by the square yard of coating in place on concrete pile surfaces. No separate payment will be made for primer.

**Basis of Payment**

The accepted quantities of bitumen coating will be paid for at the contract unit price per square yard, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the Engineer.

Payment will be made under:

| Pay Item | Pay Unit |
|---|---|
| Bitumen Coating | Square Yard. |

# APPENDIX H

**Example Specification for Bitumen Coating**

**Description**

This work shall consist of furnishing and applying bituminous coating and primer to steel pile surfaces as required in the plans and as specified herein.

**Materials**

A. Bituminous Coating. Canal Liner Bitumen (ASTM D-2521) shall be used for the bitumen coating and shall have a softening point of 190 degrees F., to 200 degrees F., a penetration of 56 to 61 at 25 degrees C., and a ductility at 25 degrees C., in excess of 3.5 cm.

B. Primer. Primer shall conform to the requirements of ASTM D-41.

**Construction Requirements**

All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather, nor when the temperature is below 65 degrees F.

Application of the prime coat shall be with a brush or other approved means and in a manner to thoroughly coat the surface of the piling with a continuous film of primer. The purpose of the primer is to provide a suitable bond of the bitumen coating to the pile. The primer shall set thoroughly before the bitumen coating is applied.

The bitumen should be heated to 300 degrees F., and applied at a temperature between 200 degrees F., to 300 degrees F., by one or more mop coats, or other approved means, to apply an average coating depth of 3/8 inch. Whitewashing of the coating may be required, as deemed necessary by the engineer, to prevent running and sagging of the asphalt coating prior to driving, during hot weather.

Bitumen coated piles shall be stored immediately after the coating is applied for protection from sunlight and heat. Pile coatings shall not be exposed to damage or contamination during storage, hauling, or handling. Once the bitumen coating has been applied, the contractor will not be allowed to drag the piles on the ground or to use cable wraps around the pile during handling. Pad eyes, or other suitable devices, shall be attached to the pile to be used for lifting and handling. If necessary, the contractor shall recoat the piles, at his expense to comply with these requirements.

A nominal length of pile shall be left uncoated where field splices will be required. After completing the field splice, the splice area shall be brush or mop coated with at least one coat of bitumen.

**Method of Measurement**

Bitumen coating will be measured by the linear foot of coating in place on the pile surfaces. No separate payment will be made for primer or coating of the splice areas.

**Basis of Payment**

The accepted quantities of bitumen coating will be paid for at the contract unit price per linear foot, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the Engineer.

Payment will be made under:

| Pay Item | Pay Unit |
| --- | --- |
| Bitumen Coating | Square Yard. |